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## ABSTRACT

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Work Package 6 (WP6) addresses integrated solutions for holistic risk reduction in Critical Infrastructures (CIs). Transportation, hydraulic and energy/telecommunication infrastructures are treated accounting for their different peculiarities. To this end, effective and efficient maintenance strategies play a crucial role among research activities carried out within WP6.

In this context, predictive maintenance methods (PMMs) are addressed in detail in Task 6.1, i.e., with the aim of developing an IT framework for the on-site implementation of such methods.

DV6.1a represents the first Section of a two-parts document regarding research advances on PMMs applied to CIs. In particular, suitable statistic methods will be presented with respect to reinforced (and prestressed) concrete, steel and composite bridges.

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### 3. Introduction

#### 3.1 Background and motivation

Critical infrastructures, e.g. transport, industrial and power grid infrastructures are essential for ensuring the safety and socio-economic growth of communities. Nevertheless, existing CIs are often outdated and vulnerable. In past times, most efforts from research and technical practice were devoted to finding suitable retrofit interventions for maintenance of damaged CIs. However, this approach proved to be occasionally ineffective as structural upgrading was only performed after damaging events (e.g., aftermaths from long-term ground motions – slow landslides, subsidence – or from extreme events such as strong earthquakes). As a result, severe losses in terms of repairing costs and temporary inoperability were sometimes achieved.

In response to such criticalities, predictive maintenance methods (PMMs) recently emerged as an efficient and effective alternative to after-damage repairing. Namely, on the basis of suitable monitoring data (i.e., through the help of dedicated SHM systems), the ideal maintenance time for CIs can be derived to balance costs and outcomes within a reliability framework (see Figure 3.1-1).

In this regard, different aspects concerning PMMs are still under investigation nowadays. For instance, while consolidated techniques are available for practitioners addressing building structures, very few indications are available in literature and/or in normative provisions with respect to linear infrastructures such as bridges. Moreover, some complex aspects such as the influence of degradation phenomena on the static and cyclic performance of CIs still need further inquiry.

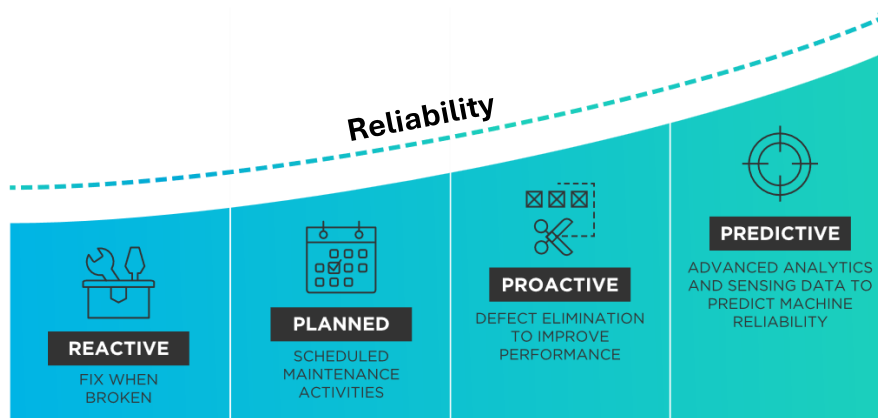


Figure 3.1-1 – Maintenance methods for Critical Infrastructures and relevant reliability level.

#### 3.2 Outline of the report

The present report is divided into three main parts. Namely, PMMs for steel and composite bridge infrastructures are addressed in Section 4. Subsequently, suitable PMMs for reinforced concrete bridges (with or without prestressing) are discussed in Section 5. Finally, main conclusive remarks drawn from dealt topics are summarized in Section 6. Each of the following Sections is articulated in relevant subSections to address the following key aspects: 1) motivation (novelty, importance of the topic), 2) Literature review (and research gaps), 3) Methodology and 4) Demonstration.

## 4. Predictive Maintenance Methods for Steel/Composite Bridges

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### 4.1 Generality

The fatigue performance of steel structures has become a relevant topic for civil engineering since the second half of the 20th century, due to the occurrence of some relevant fatigue-related failures [1,2]. In this context, steel (and composite) bridges have a crucial importance due to them being subjected to significant transient loads during service life, i.e., due to the combined action of traffic, wind and earthquake loads.

Several efforts to understand fatigue phenomenology have been made starting from the pioneering works of Wohler, leading to the publication of normative requirements for fatigue verifications of steel structures and infrastructures worldwide. With regard to the European countries, such provisions are currently codified in EN1993:1–9 [3], which provides two alternative philosophies for the fatigue checking of steel structures, namely:

- The Safe-Life (SL) approach, which does not contemplate any fatigue damage in checked structures. The SL approach is addressed by means of a punctual stress-based verification, i.e., it refers to the worst load conditions occurring in the service life;
- The Damage-Tolerant (DT) approach, which admits the development of controlled fatigue damage in verified structures. Expected damage, which is estimated accounting for the entire service life, should not exceed a threshold value associated with failure.

When dealing with existing steel infrastructures, the DT approach is clearly the most suitable option as it allows for the accounting of the damage endured by steel elements during their past service life. The amount of cumulated damage can be often significant for older structures, mainly owing to the inadequacy of the design requirements available at the erection time and/or to a progressive increase in cyclic loads over the years.

Moreover, in the case of existing steel structures a further relevant source of damage is represented by material degradation, mainly in the form of metallic corrosion [4].

Corrosion consists of progressive material loss due to an electrochemical process which is extremely sensitive to local environmental conditions (i.e., temperature, relative humidity, salinity, etc.) [5,6]. In particular, extensive portions of exposed structures can be affected by degradation or, conversely, damage can take place in highly localized spots (non-uniform corrosion) [7–10]. At the same time, the material loss rate can significantly vary during structural service life in dependence on the above parameters.

When corrosive phenomena occur in combination with cyclic loadings, corrosion-induced damage does not simply add up to fatigue damage, but rather, the two processes influence each other. Multiple factors underlie this mutual interaction, namely [11]:

- On one hand, corrosion induces a localized reduction of the resisting cross-section, resulting in stress amplifications which accelerate fatigue cracking. Moreover, the fatigue strength of iron oxide products is considerably lower in comparison to that of pristine steel;
- On the other hand, fatigue cracking creates preferable spots for corrosion development as the cracks can penetrate through protective layers, if present.

It should be remarked that both phenomena are characterized by a significant degree of randomness [1,2,5,9,10], which further intensifies when fatigue and corrosion damage are combined. To address

this aspect, two main approaches have been followed in the scientific literature, namely (i) implementing stochastic procedures to account for both fatigue and corrosion intrinsic randomness and (ii) using simplified techniques to deal with corrosion fatigue in a deterministic way, usually based on experimental data. Nevertheless, it is clear that the phenomenology of corrosion fatigue in steel structures is a complex topic which still represents an open field of research nowadays [12–15].

Therefore, it is not surprising that no codified procedures for the corrosion fatigue assessment of steel constructions are available at present time. This lack of provisions is indeed crucial as steel structures, which are conceived to endure relevant cyclic loads during their service life, are often placed in aggressive environments [16]. Nevertheless, some contributions dealing with corrosion fatigue analysis of steel structures can already be found in the scientific literature (see e.g. [17-23]).

Within this framework, in this Chapter, both diagnostic-based and analytic approaches for PM of steel bridges are presented.

Namely, in Section 4.2 and relevant subSections, a framework for performance upgrading of steel CIs based on data acquisition, diagnostics and decision making is first presented.

Subsequently, some analytical approaches for PM of steel CIs are presented (Sections 4.3-4.4). Namely, an approximated, yet deterministic technique is first introduced, i.e., based on a reformulation of fatigue checks in an explicit “demand vs. capacity” form in line with the principles of performance-based engineering. For this purpose, the concept of “critical corrosion degree”  $\eta^*_{Rd}$ , associated with an assigned target fatigue life  $t^*$ , will be introduced [24]. Subsequently, a more suitable statistic approach – based on some simplified assumptions – will be presented with the aim of deriving a time-evolving corrosion fatigue fragility (CFF) curve, i.e., thus enabling the estimation of the most suitable maintenance time  $\bar{t}$  via the reliability index  $\beta$  as defined in EN1990 [25].

Further details are provided in following subSections.

## 4.2 Service life extension of strategic bridge structures

As mentioned in the introductory Section, European and worldwide bridges are subject to rapid degradation due to external factors like environmental effects and over-usage [26-27]. Furthermore, the immediate replacement of existing bridges that ended their design life would be unsustainable in monetary terms, i.e., something which is also exasperated by the constant instability of the global economics, as strongly highlighted by recent pandemic and war events [28].

In this context, an appealing and economically valid strategy for the management of bridges that overpassed their design life, is the extension of their service life. Extending the service life of a bridges is a non-trivial operation as it implies a set of crucial actions, which must be put in place relying on a mix of expert judgment and advanced technologies:

- i) A careful evaluation of the degradation state and of the safety levels of the bridge must be carried out regularly, in proximity of the end of its service life, and in the aftermath of an impacting external event like an earthquake [29];
- ii) Appropriate monitoring/surveillance systems must be installed on the bridge to continuously map its status [30-31] and service, i.e., in order to reach a rapid diagnosis for any anomaly or damage having non-negligible impacts on the structural status and safety level;
- iii) Ordinary and special inspection and maintenance activities must be designed a-priori, and referred to each possible scenario;

- iv) Bridge modelling must be carefully carried out with the aid of some dedicated techniques (e.g., putting in place a blind modelling activity), to double-check the results coming from the structural analyses, which are taken as basis of the decision-making process.

In this sense, the service life extension must be intended as a special operational regime where the above-mentioned crucial actions lead to something that can be defined a “structural augmentation” of the bridge, taking inspiration from what is defined as “augmented reality” [32].

The “augmented bridge structure” can be intended as a bridge structure in which systemic qualities (e.g. structural health monitoring, surveillance, digital twin of the bridge) are introduced to balance or control structural deficiencies, thus allowing the real capacity of the bridge to be better exploited. In this way, the structure is hence “increased” (or “augmented”) with respect to the prior situation.

The structural augmentation for service life extension is a strategy that is particularly suitable for those bridges which are strategic for the transportation network [33]. In these cases, the value of the single bridge cannot simplistically refer to its (direct) construction cost, but it must be extended and completed with indirect aspects such as:

- i) the cost at the end of its service life, which includes both the construction cost and the costs of subsequent maintenance;
- ii) the value of the bridge in the context: indeed, an interruption of transit can involve very high costs, i.e., often higher than the direct cost of the bridge itself. When traffic is diverted to other links of the network, often inadequate to support increases in transit, significant penalties for the flow can occur.

For these reasons, for some critical bridges, the economic loss due to the interruption of transit for a few months can exceed the cost of the bridge itself by several times. Another crucial aspect related to the service life extension of a strategic bridge, concerns the involvement of public safety and potential social impacts of a collapse, making the role of political decisions relevant to the matter [34]. In summary, due to economical and sustainability aspects, strategic bridges need to stay as long as possible in service with no (or minimal) traffic interruptions.

A framework for service life extension of bridges should be based on the problem framing shown in the flowchart depicted in Figure 4.2-1. When the service life of the bridge approaches its design life, three macro-components should be put in place to pertinently take the decision of extending its service life as a structural augmented bridge: *i)* Data acquisition, *ii)* Diagnostics and *iii)* Strategy and decisions. Such components are discussed in the following subSections.

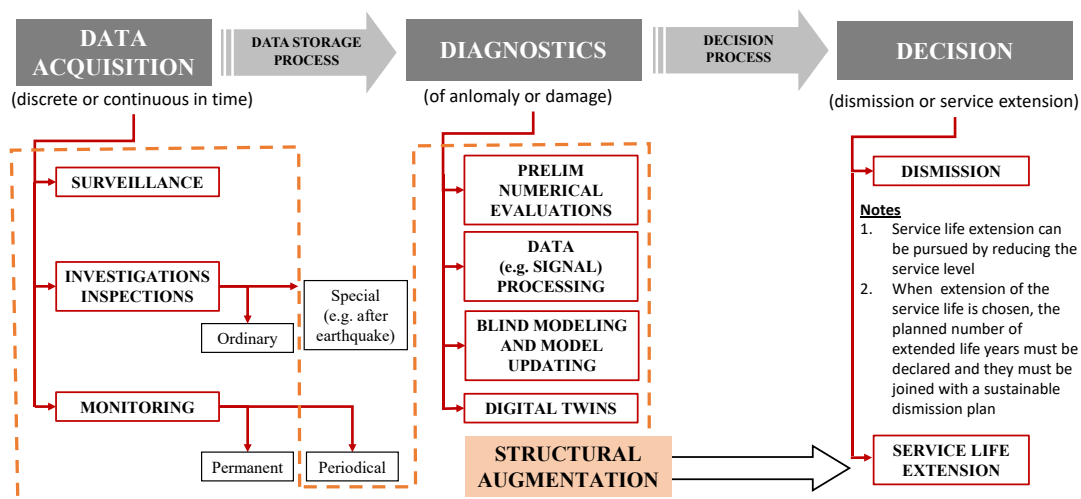


Figure 4.2-1. Framework for service life extension of bridges.

#### 4.2.1 Data acquisition

The data acquisition process for a bridge in service involves all activities required to assess its structural condition and operational status. The first step is surveillance, which includes intelligent monitoring actions (e.g., video recording, traffic control, access management) to document service conditions, usage, and weather data. Surveillance is essential both for forensic analysis in the event of failures (as in the case of the 2018 Morandi bridge collapse) and for verifying maintenance history. Alongside surveillance, visual inspections and non-destructive testing (NDT) should be conducted to validate monitoring data and assess the physical state of critical components (e.g., bearings, joints) and the mechanical properties of the structure. The third key component is instrumental monitoring, typically via a Structural Health Monitoring (SHM) system, which enables real-time assessment and long-term tracking of the bridge's behavior.

To effectively support structural enhancement and decision-making, a robust monitoring strategy should include:

- Knowledge of the original structural design and test data;
- Identification of vulnerable components prone to failure mechanisms;
- Scheduled visual inspections complementing instrumental data;
- A priori selection of measurable quantities, ensuring they are meaningful and theoretically interpretable;
- Defined procedures for data acquisition, processing, transmission, and storage;
- Implementation of advanced post-processing techniques to extract useful insights from large datasets;
- Minimization and continuous validation of sensor systems;
- Availability of a numerical model for comparison;
- Periodic reporting to track structural evolution and maintenance needs.

Finally, to ensure continuous operation and reliability, the system must include uninterrupted power supply for sensors and cameras, and adopt industrial-grade maintenance protocols, which are more demanding than typical civil engineering standards.

#### 4.2.2 Diagnostics

When outputs coming from the data acquisition activities are available, they must be correctly elaborated to provide accurate diagnostics of any anomaly or damage that can occur before and during the service life extension period. As said above, the univocal definition of data acquisition and storage methods is very important due to the expected amount of data coming from a permanent/continuous data acquisition system. A common example of data processing is the elaboration (e.g. Fourier transform) of the bridge deck vertical accelerations time histories registered by sensors, something that allows to identify the natural bending vibration frequencies and stiffness of the bridge: in these cases the sampling window and frequency must be determined by a sensitivity and data storage capability analysis.

One of the most important actions for diagnostic purposes is the activity concerning the numerical (usually Finite Element – FE) modelling of the bridge both for preliminary and advanced evaluations: numerical modelling can be implemented in a multi-scale philosophy [35]; starting from simplified analytical models, and gradually increasing the model details level and complexity by implementing model updating techniques, which implies the use of the experimental data to refine the model up to

the highest possible level of Digital Twin (of the bridge, which is an highly detailed and high fidelity model, automatically updated (e.g. mechanical characteristics) in real-time by the data flow coming from the data acquisition activities).

It is important to say that the digital twin level should be reached only if really needed (e.g. because the bridge is crucial for the network and is working in non-ideal conditions). In the diagnostics phase, an important activity that should be carried out, is the so-called “blind modelling” [49], which is a step where different, redundant models of the structure are developed, in parallel and separately, by different modellers. The blind modelling activity is very useful to spot any possible modelling error or uncertainty which may lead to an erroneous diagnostic, and it is definitively needed for complex structural systems.

#### 4.2.3 Decision

For various reasons, in the case of incomplete satisfaction of performance and safety requirements, a bridge or viaduct can be upgraded or replaced. Nevertheless, such an arrangement may not be feasible due to economic shortcomings or constructional complexity: in the latter case, the bridge owner may need time to organize the replacement/rehabilitation works, also due to the presence of concomitant constraints. Among these impediments, especially in urban complex area, there may be constraints related to the difficulty of rapidly setting up construction site activity in a densely populated area. Alternatively, the structural deficiencies may be limited by the structural augmentation activities described above, by extending the service life of the bridge. In these cases the sudden dismissal of the bridge appears an unnecessary economical expense. It is important to highlights that the service life extension can be pursued by reducing the service level of the bridge (e.g. by reducing the traffic load), and that when extension of the service life is chosen, the planned number of extended life years must be declared and they must be joined with a sustainable dismissal plan.

### 4.3 Approximated deterministic approach for PM of steel bridges

The proposed method for the analytical and deterministic PM of critical steel and composite bridges in aggressive environments is based on the well-known Palmgren–Miner’s rule [36] for damage cumulation, which is already codified in EN1993:1–9 within the framework of the DT approach [3].

Nevertheless, in this DV, fatigue checks will be manipulated in order to appear in a more direct “capacity vs. demand” approach within the framework of performance-based engineering. This aspect is addressed by introducing the concept of “critical corrosion degree”  $\eta^*_{Rd}$ , which is derived with respect to an assumed target fatigue life  $t^*$ .

The main stages of the presented procedure can be summarized as follows (see Figure 4.3-1):

- Step 1: Definition of target fatigue life  $t^*$  and selection of the corrosion trend according to exposition conditions (see subSection 4.3.1);
- Step 2: “Unaltered” structural analysis on the pristine structure, i.e., provisionally disregarding the effects of corrosion (see subSection 4.3.2);
- Step 3: Derivation of “equivalent” load spectrums accounting for corrosion-induced local stress amplifications (see subSection 4.3.3);
- Step 4: Estimation of critical corrosion degree  $\eta^*_{Rd}$  by means of a target function  $f(\eta, t^*)$  (see subSection 4.3.4);

- Step 5: Corrosion fatigue checks in “demand/capacity” form and subsequent estimation of the optimal maintenance time (see subSection 4.3.5).

Each of the introduced steps will be discussed in detail in the following subsections.

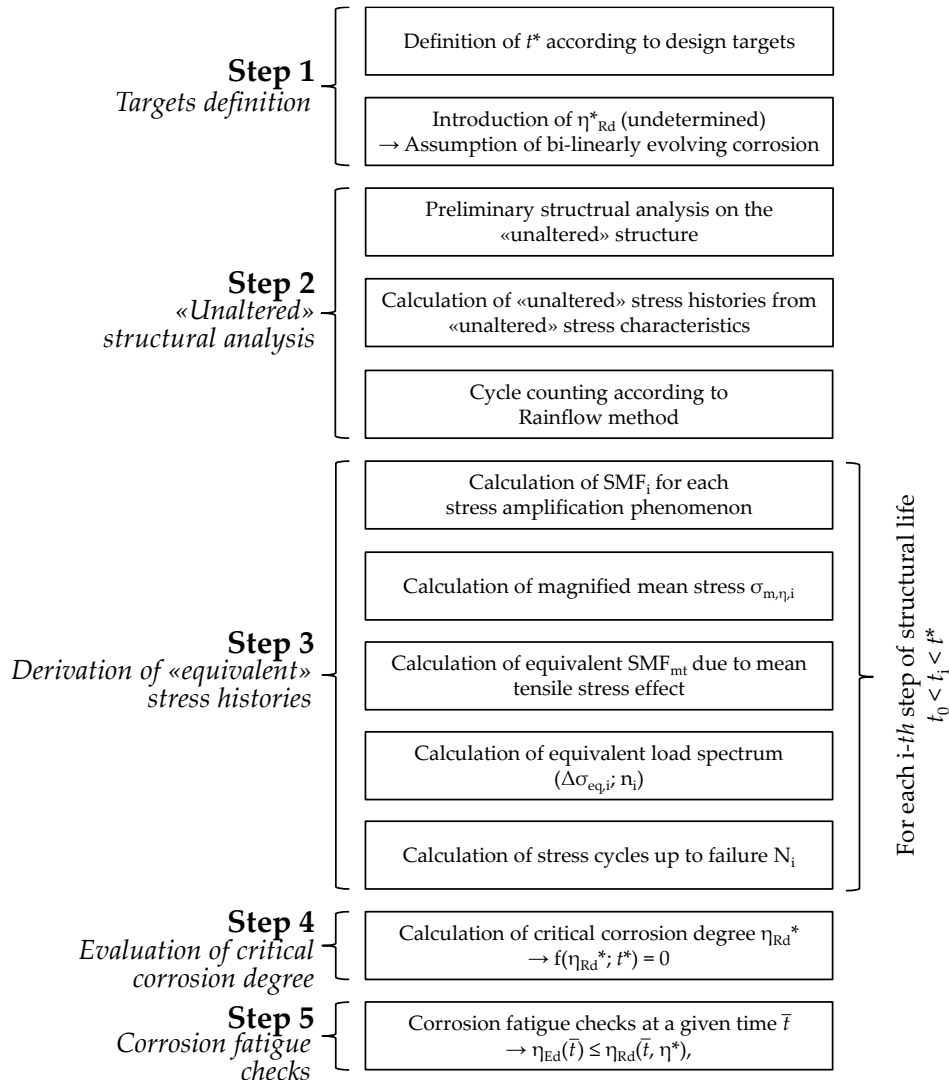


Figure 4.3-1 – Articulation of the proposed methodology for the predictive maintenance of steel and composite infrastructures in aggressive environments.

#### 4.3.1 Step 1: Definition of Target Design Life and of Corrosion Development

As highlighted in Section 4.1, the corrosive processes are extremely sensitive to local boundary conditions. In general, in the years immediately after the construction time, corrosion is limited due to the presence of preventive measures. Once the protection layer is completely worn, the corrosion process sharply accelerates.

Therefore, after significant quantities of superficial corrosion products have already formed, the corrosive process generally slows down, approaching a stabilized rate of progression (See Figure 4.3-2, dashed black curve — qualitative corrosion development) [4,9,37]. Several attempts have been made in the scientific literature to analytically describe this trend, i.e., by means of broken lines or more sophisticated power laws, which appeared particularly suitable in the case of highly localized corrosion [7,8,26].

Nevertheless, it should be remarked that the instantaneous corrosion rate  $\dot{\eta}$  (i.e., the time derivative of the corrosion degree  $d\eta/dt$  at a given time) varies sensibly through the structural service life in dependence on multiple factors (temperature, relative humidity, aggressive agent concentration, etc.). Thus, predicting the actual trend of corrosion degree against time is a complex task without the aid of a constant structural monitoring.

Moreover, even when the necessary information is available, the analysis of corrosion development is highly impractical and not free from a consistent degree of uncertainty.

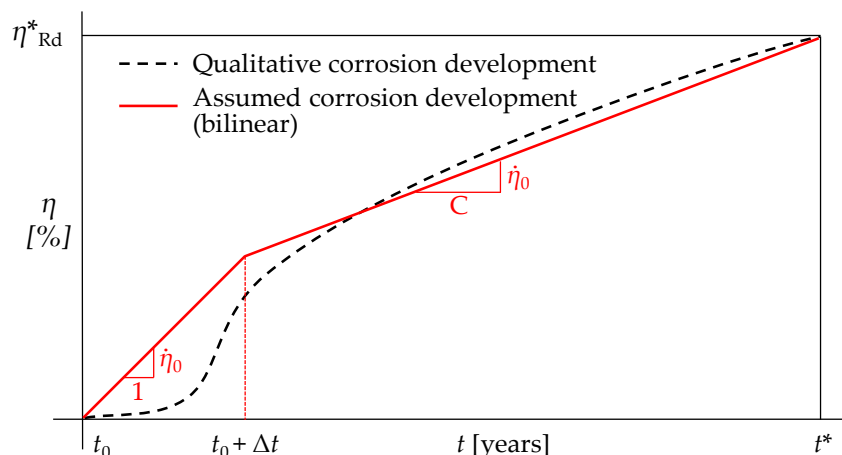


Figure 4.3-2 – Qualitative (black dashed curve) and assumed (red solid polyline) trends for corrosion development for the proposed methodological approach.

Therefore, according to prescriptions from UNI EN ISO 9224 [37], in the presented approach the corrosion development is approximated by a polyline with two branches (i.e., with a piecewise constant corrosion rate; see Figure 4.3-2, red solid line—assumed corrosion development).

The ratio among the slopes of the consecutive branches is equal to  $C : 1$ , with  $C \geq 1$  depending on the corrosivity category of concern. The suggested values of parameter  $C$  for each category are summarized in Table 4.3-1.

Table 4.3-1 – Suggested values of  $C$  for each corrosivity category according to [37].

| Corrosivity Category | Exposition | $C$ [-] |
|----------------------|------------|---------|
| C1                   | Very low   | 5       |
| C2                   | Low        | 3.5     |
| C3                   | Medium     | 2.5     |
| C4                   | High       | 1.5     |
| C5                   | Very high  | 1       |

The knee point of the bi-linear curve is assumed to occur  $\Delta t$  years after construction time  $t_0$ . According to the recommendations from [37], in the absence of more precise information,  $\Delta t = 10$  years should be selected.

The corrosion degree  $\eta$  is assumed null for  $t = t_0$ . Conversely, in correspondence with the target fatigue life  $t^*$ , a “critical corrosion degree”  $\eta^*_{Rd}$  is introduced, i.e., the minimum corrosion degree which exactly induces corrosion fatigue collapse for the assumed  $t^*$ .

It should be remarked that this assumption, i.e., monotonically increasing  $\eta(t)$ , can be considered sufficiently accurate in the case of scarcely maintained structures, while it will lead to conservative estimations in the case of properly preserved structures as protective measures such as coatings and polishes can temporarily arrest corrosion development [4,10].

The selection of a proper value for  $t^*$  has to be intended as a designer's choice, which depends on the expected influence of both the fatigue and the material degradation on structural performance. Nevertheless, owing to the sudden and undesirable nature of corrosion fatigue failures,  $t^*$  should always be sufficiently greater than the design's working life  $L_{wd}$ . For this purpose, an appropriate fatigue life multiplier  $k > 1$  can be introduced, i.e., the ratio of  $t^*$  and  $L_{wd}$ .

Hence, in the absence of more detailed information, for structures in which fatigue already governs the design choices, smaller values of  $k$  can be assumed (that is,  $k = 1.1 \div 1.2$ ). Conversely, for structures in which fatigue collapse is not expected and/or for which a significant ultimate ductility is required, higher values of  $k$  should be selected (that is,  $k = 1.5 \div 2.0$ ). Nevertheless, it should be remarked that more accurate values of  $k$  can be derived on a case-by-case basis according to reliability considerations, in compliance with the EN1990 provisions [38].

Consistently with the assumption of a piecewise constant corrosion rate, the corrosion degree at a given time  $\eta(t)$  within the service life is expressed by Equations (1)-(2):

$$\eta(t) = \begin{cases} \dot{\eta}_0(t - t_0) & t \leq t_0 + \Delta t \\ \frac{\dot{\eta}_0}{c}(t - t_0) + \Delta t \dot{\eta}_0 \left(1 - \frac{1}{c}\right) & t_0 + \Delta t < t \leq t^* \end{cases} \quad (1)$$

$$\dot{\eta}_0 = \frac{\eta_{Rd}^*}{\Delta t \left(1 - \frac{1}{c}\right) + \frac{t^*}{c}} \quad (2)$$

in which  $\dot{\eta}_0$  represents the corrosion rate assumed for the first branch of the polyline, which, for a given corrosivity category, is only a function of  $\eta_{Rd}^*$  and  $t^*$  (see Equation (2)). Equations (1) and (2) hold true for an arbitrary value of  $\eta_{Rd}^*$ , which is still undetermined in Step 1. The estimation of the actual value of  $\eta_{Rd}^*$  will be addressed in Step 4 (see subSection 4.3.4 for further details).

#### 4.3.2 Step 2: “Unaltered” Structural Analysis

A common assumption made in the global analysis of corroded structural members is to neglect any self-weight and stiffness reduction due to mass loss [10,12]. This assumption can be considered sufficiently accurate since, according to [37], the expected values of material loss do not exceed a few mg/(year·m<sup>2</sup>), even in the case of extremely aggressive environments (i.e., for corrosivity category C5). Moreover, in most circumstances neglecting stiffness degradation leads to a conservative estimation of the stress characteristics in corroded structural members. Indeed, while no stress redistribution is expected for isostatic damaged structures, in the case of redundant structures corrosion would result in stress migration towards the intact elements.

Therefore, structural analysis may be performed considering the “unaltered” structure in order to evaluate the values of the stress characteristics ( $S_0$ ) only once. The influence of corrosion in terms of local stress amplifications will be considered for a second time by means of the appropriate magnification factors.

The results of “unaltered” structural analysis should account separately for the permanent and transient load as in most cases the former single-handedly contributes to the definition of minimum stresses in the structural elements ( $S_{0,min}$ ), while the latter induces stress fluctuations ( $\Delta S_0$ ) influencing the maximum values of the stress characteristics ( $S_{0,max}$ ).

#### 4.3.3 Step 3: Evaluation of “Equivalent” Corrosion-Depending Load Spectrums

Once the “unaltered” stress characteristics  $S_{0,min}/\Delta S_0$  have been determined, the “unaltered” local stresses can be determined using the well-known expressions from the theory of elasticity.

Therefore, for each time during the service life, the “real” corrosion-depending stresses  $\sigma_{\eta,\min}/\Delta\sigma_{\eta}$  can be determined by amplifying  $\sigma_{0,\min}/\Delta\sigma_0$  by the means of stress magnification factors (SMFs).

In general, when dealing with steel structures in aggressive environments, at least two sources of stress amplification should be considered, i.e., the local reduction of the resisting cross-section induced by corrosion and the presence of mean tensile stresses  $\sigma_{0,m} > 0$  under pulsating loads [39]. More properly, the latter condition results in a reduction in fatigue strength rather than an increase in fatigue demand, although several studies have highlighted that the mean stress effect can be accounted for by amplifying the design stress range up to an equivalent value [40–42].

Corrosion-induced amplification can be modelled by means of an SMF (henceforth referred to as  $SMF_{\eta}$ ), which is a function of (i) the pristine resisting cross-section properties  $X_0$  (in which  $X$  can stand for area, second moment of area, etc., depending on the monitored stress characteristics); (ii) the cross-section properties reduction  $\Delta X_{\eta}$ ; and, generally speaking, (iii) the nature of the corrosive process ( $CP$ ), as follows (Equation (3)):

$$SMF_{\eta} = f(X_0, \Delta X_{\eta}, CP, \eta_{Rd}^*, t) \geq 1 \quad (3)$$

Indeed, as corrosive processes depend on local boundary conditions, corrosion can induce strongly localized damage in small portions of the structural members in the form of sharp “pits” [5,9]. This phenomenon, which is known as “pitting corrosion”, can induce local stress magnifications which overly exceed the values estimated for uniform corrosion, i.e., the condition in which material loss is homogenously spread along the element. Uniform corrosion represents the most favorable condition for damaged steel structures as stress amplifications attain a minimum value, which can be expressed as follows (Equation (4)):

$$SMF_{\eta,uniform} = \frac{1}{1 - \frac{\Delta X_{\eta}}{X_0}} = \min_{CP}(SMN_{\eta}) \quad (4)$$

“Real” corrosion-depending stresses  $\sigma_{\eta,\min}/\Delta\sigma_{\eta}$  are hence derived as follows (Equations (5)-(6)):

$$\sigma_{\eta,\min} = SMF_{\eta} \sigma_{0,\min} \quad (5)$$

$$\Delta\sigma_{\eta} = SMF_{\eta} \Delta\sigma_0 \quad (6)$$

As in most cases “unaltered” stress histories due to transient loads are already complex and aperiodic, it is clear that “real” stress histories cannot be described by a single value  $\Delta\sigma_{\eta}$ , but should, rather, be expressed in the form of “real” oscillograms, in which each point  $\sigma_{\eta}(t)$  is derived by the amplifying of  $SMF_{\eta}$  times the related “unaltered” stress  $\sigma_0(t)$ .

At this point, cycle counting is performed considering only the fluctuating part of the amplified stresses  $\Delta\sigma_{\eta}$ . For this purpose, both the Reservoir and the Rainflow methods (which are codified in EN:1993-1-9 [3]) can be used, although the latter procedure is more suitable in light of a computational implementation of the presented methodological approach [2].

Cycle counting yields a first “approximation” of the corrosion-depending load spectrum ( $\Delta\sigma_{\eta,i}; n_i$ ). The equivalent fatigue demand  $\Delta\sigma_{eq,i}$  is finally estimated considering the effect of the mean tensile stresses, as follows (Equation (7)):

$$\Delta\sigma_{eq,i}(t) = \Delta\sigma_{\eta,i}(t) \cdot SMF_{Eq}(t) \quad (7)$$

where  $SMF_{Eq}$  is an equivalent magnification factor accounting for the mean stress effect. According to the experimental evidence and consolidated practice in fatigue analysis [2], Goodman’s model [40] is selected in this work to deal with pulsating stress histories.

It should be remarked that  $SMF_{Eq}$  implicitly depends on  $\Delta\sigma_{\eta,i}$ , as Equations (8)-(9) hold true:

$$SMF_{Eq}(t) = \frac{1}{1 - \frac{\sigma_{\eta,m,i}(t)}{f_u}} \quad (8)$$

$$\sigma_{m,\eta,i}(t) = \sigma_{\eta,min,i}(t) + \frac{\Delta\sigma_{\eta,i}(t)}{2} \quad (9)$$

with  $f_u$  being the ultimate tensile strength (UTS) of structural steel.

As shown by González-Arévalo et al. [43], steel properties can actually deteriorate in time due to material aging, resulting in accelerated damage evolution. Specifically, up to a 4% decrease in UTS can be found in aged steels with respect to the original value. Therefore, in the case of structures conceived to endure for a very long service life, this effect can be accounted for by introducing a time dependence for  $f_u$ .

In this way, accelerated damaging due to steel aging can be equivalently considered, i.e., with a further increase of  $SMF_{Eq}(t)$  in comparison to the value deriving from the lone mean stress effect. The evaluation of  $\Delta\sigma_{eq,i}$  is performed for each stage of the structural service life, thus obtaining  $(t^* - t_0)$  “equivalent” load spectrums, which represent the overall fatigue demand on the analyzed steel structure in an aggressive environment (Figure 4.3-3).

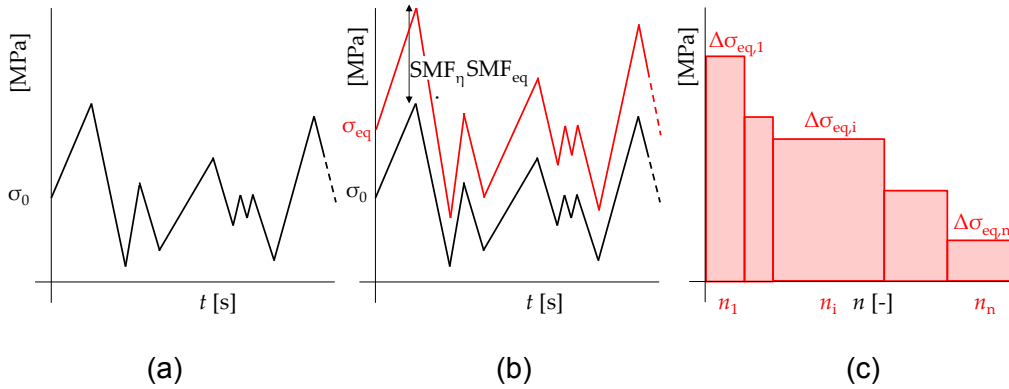


Figure 4.3-3 – Estimation of “real” load spectrum from aperiodic stress histories according to the proposed methodological approach: (a) “unaltered” stress oscillograms, (b) “real” and “equivalent” stress oscillograms, and (c) “equivalent” load spectrums.

#### 4.3.4 Step 4: Evaluation of Critical Corrosion Degree $\eta^*_{Rd}$

Using Palmgren–Miner’s rule for linear damage cumulation, the actual value of the critical corrosion degree  $\eta^*_{Rd}$  can be estimated by imposing that the total damage parameter  $D_{TOT}$  reaches unity for  $t = t^*$ . This operation can be performed numerically by solving the following implicit equation for  $\eta^*_{Rd}$  (Equation (10)):

$$f(\eta^*_{Rd}, t^*) = 1 - D_{TOT}(\eta^*_{Rd}, t^*) = 1 - \sum_{t=t_0}^{t^*} \frac{n_i}{N_i(\Delta\sigma_{eq,i}(\eta^*_{Rd}))} - 1 = 0 \quad (10)$$

The numerical solution of Equation (10) only requires a few iterations as  $D_{TOT}$  monotonically decreases for increasing values of  $\eta^*_{Rd}$  (See Figure 4.3-4).

In order to account for fatigue strength reduction due to material degradation, the evaluation of the number of stress cycles up to failure  $N_i$  has to be performed using a S-N- $\eta$  fatigue strength domain in place of the S-N curves provided by EN1993:1–9 [3].

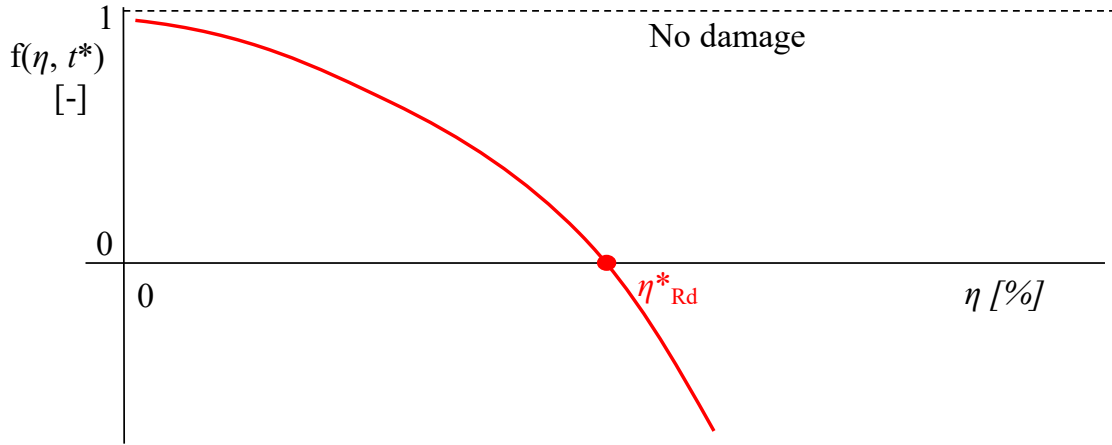


Figure 4.3-4 – Monotonic trend of  $f(\eta, t^*)$  and corresponding value of  $\eta^*_{Rd}$  for  $f(\eta, t^*) = 0$ .

On the basis of the results reported in [19,21,44,45], the following expressions can be used (Equations (11)-(12)):

$$\log \frac{5 \times 10^6}{N_i} = m_{1,\eta} \log \left( \frac{\Delta \sigma_{D,\eta}}{\Delta \sigma_{Eq,i}} \right) \quad (11)$$

$$\log \frac{N_i}{5 \times 10^6} = m_{2,\eta} \log \left( \frac{\Delta \sigma_{Eq,i}}{\Delta \sigma_{D,\eta}} \right) \quad (12)$$

with  $\Delta \sigma_{D,\eta}$  being the modified constant amplitude fatigue limit (CAFL) for the selected structural member, accounting for the effect of corrosion (i.e., the stress range which induces corrosion fatigue failure for  $N = 5 \times 10^6$ ), and  $m_{1,\eta}/m_{2,\eta}$  being the corrosion-depending inverse slopes of the LCF and HCF branch of the S-N curve, respectively.

According to [21,45], the modified CAFL can be derived assuming that very low-cycle fatigue (VLCF) behavior is unaffected by corrosion, i.e., the stress range  $\Delta \sigma_{10000}$  inducing fatigue collapse for  $N = 10^4$  does not depend on  $\eta$ . Thus,  $\Delta \sigma_{D,\eta}$  can be calculated through Equation (13):

$$\Delta \sigma_{D,\eta} = \frac{\Delta \sigma_{10000}}{(5 \times 10^2)^{\frac{1}{m_{1,\eta}}}} \quad (13)$$

$\Delta \sigma_{10000}$  is immediately derived from the detail class for the structural member  $\Delta \sigma_c$ , i.e., the stress range causing fatigue collapse for  $N = 2 \times 10^6$  in the absence of corrosion (Equation (14)):

$$\Delta \sigma_{D,10000} = (2 \times 10^2)^{\frac{1}{m_{1,0}}} \cdot \Delta \sigma_c \quad (14)$$

For the sake of clarity,  $m_{1,0}$  is the inverse slope of the LCF branch according to EN1993:1–9 [3] in pristine conditions. Consistently with the results reported in [19,22], the inverse slopes are assumed to linearly reduce as  $\eta$  increases, as follows (Equations (15)-(16)):

$$m_{1,\eta} = m_{1,0} - \frac{\eta}{\eta_{ref}} \Delta m_{1,\eta} \quad (15)$$

$$m_{2,\eta} = m_{2,0} - \frac{\eta}{\eta_{ref}} \Delta m_{2,\eta} \quad (16)$$

in which  $m_{1,0}/m_{2,0}$  are the inverse slopes of the S-N curve for LCF/HCF in pristine conditions, respectively;  $m_{1,\eta}/m_{2,\eta}$  are the corrosion-affected inverse slopes for the same branches; and  $\eta_{ref}/\Delta m_{1,\eta}/\Delta m_{2,\eta}$  are the experimental parameters expressing the influence of corrosion on the fatigue behavior of the steel components.

According to the results from [21,45],  $\eta_{\text{ref}} = 0.2$ ,  $\Delta m_{1,\eta} = 0.375 m_{1,0}$ , and  $\Delta m_{2,\eta} = 0.375 m_{2,0}$  are, hence, assumed. In this way, the observed increase of the slopes (i.e.,  $\approx 1.6$  times their original values) is attained for the mean value of corrosion detected in the reference experimental tests (i.e.,  $\eta_{\text{mean}} \approx 20\%$ ).

Conservatively, and in line with the experimental evidence [10,39,46], for the corrosion fatigue domain no endurance limit is assumed, i.e.,  $m_{2,\eta} < \infty$ . The resulting shape of the S-N- $\eta$  domains, according to Equations (11)–(16), is depicted in Figure 4.3-5 for increasing levels of  $\eta$ .

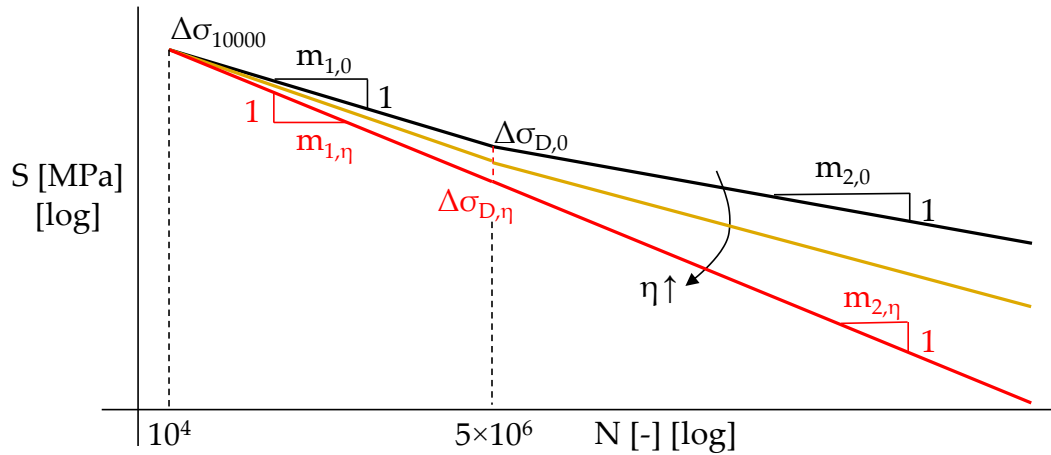


Figure 4.3-5 – Assumed shape of S-M- $\eta$  domains for corrosion fatigue analyses.

#### 4.3.5 Step 5: Corrosion Fatigue Checks

Once the value of  $\eta_{\text{Rd}}^*$  is known, the corrosion fatigue checks are immediately performed by controlling that “corrosion demand” at a given time  $\eta_{\text{Ed}}(\bar{t})$  does not exceed the “corrosion capacity”  $\eta_{\text{Rd}}(\bar{t})$  for the same  $\bar{t}$ , which is simply derived from  $\eta_{\text{Rd}}^*$  according to the assumption of the piecewise constant corrosion rate (Equation (17)):

$$\eta_{\text{Ed}}(\bar{t}) \leq \eta_{\text{Rd}}(\bar{t}) = \eta_{\text{Rd}}^* - \frac{\eta_{\text{Rd}}^*}{\Delta t(C - 1) + t^*} (t^* - \bar{t}) \quad (17)$$

The main strength of the presented approach lies in the fact that  $\eta_{\text{Rd}}^*$  is a constant reference value which can be easily determined numerically, thus allowing the calculation  $\eta_{\text{Rd}}(\bar{t})$  for a generic instant of assessment  $\bar{t}$ , while  $\eta_{\text{Ed}}(\bar{t})$  can be directly estimated on the basis of on-field monitoring evidence. On the other hand, significant uncertainties regarding load histories (for example, in the case of wind-induced fatigue in aggressive environments) or corrosion trends which sensibly differ from the assumed expression for  $\eta(t)$  (see Equation (1) for example, in the case of periodically maintained structures) may negatively affect the reliability of the presented procedure.

On the basis of the results of the corrosion fatigue checks, three different damage stages can be identified for the investigated structure, namely (Figure 4.3-6a):

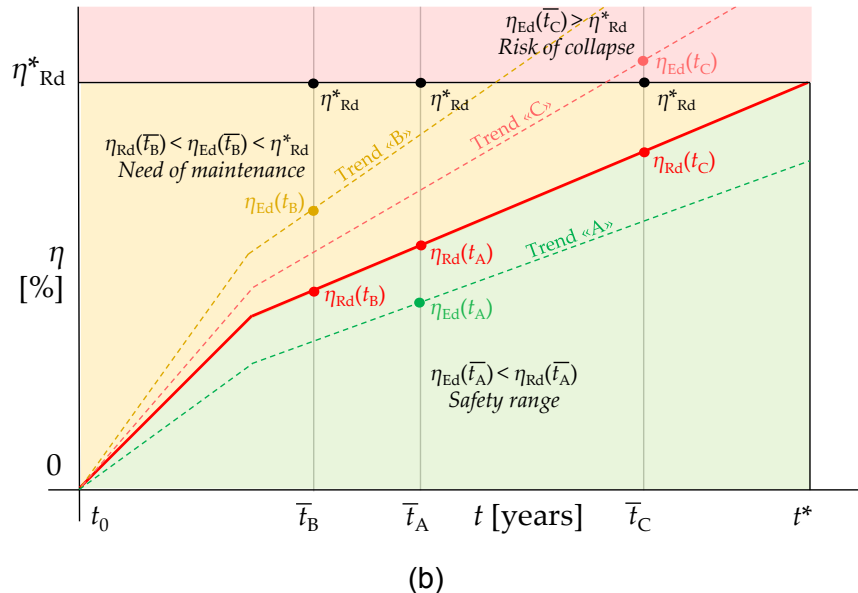
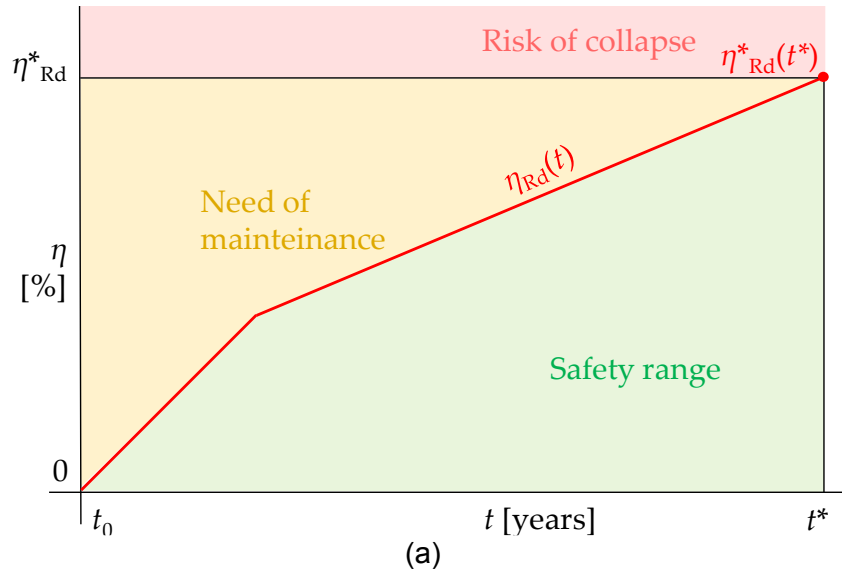


Figure 4.3-6 – (a) Damage stages due to corrosion fatigue according to the presented methodological approach and (b) examples of fatigue check results for different corrosion trends and instants of assessment.

- For  $0 < \eta_{Ed}(t) < \eta_{Rd}(t)$ , the structure can be considered safe with regard to corrosion fatigue failure. Nevertheless, periodic inspections should be carried out to verify if this condition holds true in time (Figure 4.3-6b, Trend “A”);
- For  $\eta_{Rd}(t) < \eta_{Ed}(t) < \eta^*_{Rd}$ , the necessity of maintenance measures emerges as values of  $\eta > \eta^*_{Rd}$  are expected to be attained prior to the target fatigue life  $t^*$  (Figure 4.3-6b, Trend “B”);
- For  $\eta_{Ed}(t) > \eta^*$ , the structure quickly requires safety measures as risk of collapse due to corrosion fatigue is predicted (Figure 4.3-6b, Trend “C”).

## 4.4 Probabilistic approach for PM of steel bridges

In order to decline analytical PM of steel bridges in a more proper probabilistic framework, some additional assumptions have to be introduced, i.e., in order to derive a corrosion fatigue fragility

(CFF) curve for CIs. This will enable the estimation of the most suitable maintenance time  $\bar{t}$  via the well-known reliability index  $\beta$  as defined in EN1990 [25].

In the following subSections, the proposed methodology will be presented with first reference to a steel bridge affected by lone fatigue issues, i.e., as in this case a closed-form fatigue fragility (FF) curve can be derived (see §4.4.1). Subsequently, the extension of such methodology with reference to corrosion fatigue issues will be discussed (see §4.4.2).

#### 4.4.1 Fatigue fragility for steel bridges

The proposed method for analytic derivation of FF in absence of corrosive phenomena is based on few simplified assumptions, i.e. (see also Figure 4.4-1):

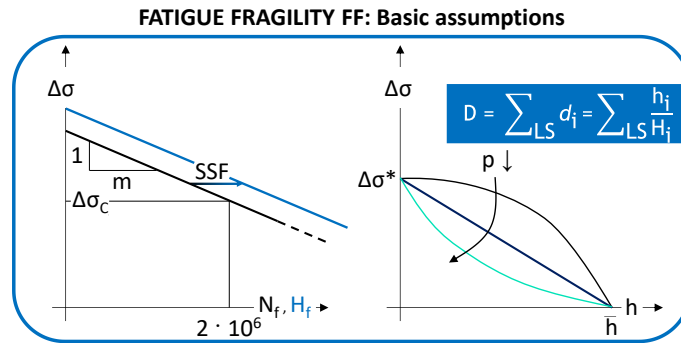


Figure 4.4-1 – Graphical representation of basic assumptions for the estimation of FF according to the proposed approach.

- The fatigue strength domain (S-N curve) for a given detail can be expressed via a 2-parameter (A, b) Basquin's Law (Equation (18)):

$$N_f = A \Delta\sigma^{-b} \quad (18)$$

For a given detail, S-N curves for a different confidence interval are parallel to each other. Randomness of the S-N curve can be hence expressed through a single error parameter  $\varepsilon$  with null expected value and variance estimated based on experimental data;

- Fatigue strength for variable-amplitude loads can be analytically derived from the Wohler curve, i.e., provided that it is expressed with respect to cumulated cycles  $H_f = \sum_i N_i$  via the so-called spectrum shape factor SSF as defined by Gassner (Equation (19)) [46]:

$$\begin{cases} H_f = \text{SSF} \cdot A(P_F) \Delta\sigma^{-b} \\ H_f = \sum_i N_i \end{cases} \quad (19)$$

where  $\text{SSF} \geq 1$  accounts for the load spectrum (LS) shape (i.e.,  $\text{SSF} = 1$  is achieved only for a constant-amplitude fatigue loading). As shown by Navrátil [47], typical values of SSF for steel bridges are in the range  $2 \div 3$ , while the Gassner curve slope being still equal to  $b$ .

- LS for the detail of concern can be expressed through a 3-parameter  $(\Delta\sigma^*, \bar{h}, p)$  expression as follows (Equation (20)):

$$\left( \frac{\Delta\sigma}{\Delta\sigma^*} \right)^p = 1 - \frac{\log_{10} h}{\log_{10} \bar{h}} \quad (20)$$

where  $h = \sum_i n_i$ ,  $\Delta\sigma^*$ ,  $\bar{h}$  are reference  $\Delta\sigma$ ,  $h$  values, and  $p$  governs the LS shape;

- Damage cumulation due to variable-amplitude fatigue can be addressed by extending Miner's rule to the cumulated cycles' domain  $H_f-N$ .

On the basis of the above assumptions, fatigue fragility FF can be expressed via the following conditioned probability  $P(\text{Failure} \mid \text{Parameters})$  (Equation (21)):

$$FF = P(D \geq D^* \mid \Delta\sigma^*, p, b, \bar{h}, t) \quad (21)$$

where  $D$  is the total fatigue damage estimated through integration of infinitesimal damage over the LS (continuous Miner's rule), while  $D^*$  is the damage threshold for which fatigue failure is assumed. As reported in [48],  $D^*$  can be assumed as a random variable with unitary mean and 30% variance.

Moreover, an explicit dependence on time  $t$  is introduced to estimate FF at any given time during service life, i.e., through a normalized coefficient  $k(t)$ . By replacing Equations (18)-(20) into Equation (21), the following expression for FF can be derived (Equation (22)):

$$FF = \Phi \left\{ \frac{1}{s} \log \left[ C^* \frac{k(t)}{A} \left( \frac{\Delta\sigma^*}{\log h^{1/p}} \right)^b \bar{h} \right] \right\} \quad (22)$$

where  $s$  accounts for the combined standard deviation of  $H_f-N$  curve and damage threshold  $D^*$ ,  $k(t)$  is defined as the ratio among the journeyed traffic JT in the period  $t_0 \div t$  and the total traffic TT in the entire analysis window and  $C^*$  is a convenient constant depending on  $p$ ,  $b$ .

$C^*$  values for most common values of  $p$ ,  $b$  are provided in Table 4.4-1. Examples of fatigue fragility curves estimated for different values of conditioning parameter is depicted in Figure 4.4-2, i.e., by assuming  $\Delta\sigma^*$  as the intensity measure of concern.

Table 4.4-1 – Values of constant  $C^*$  to estimate FF for typical values of  $p$ ,  $b$ .

| LS shape parameter $p$ [-] | Fatigue domain slope $b$ [-] | Constant $C^*$ [-] |
|----------------------------|------------------------------|--------------------|
| 1.5                        | 3.0                          | 2.1                |
| 1.5                        | 5.0                          | 9.7                |
| 2.0                        | 3.0                          | 1.4                |
| 2.0                        | 5.0                          | 3.5                |
| 3.0                        | 3.0                          | 1.0                |
| 3.0                        | 5.0                          | 1.6                |

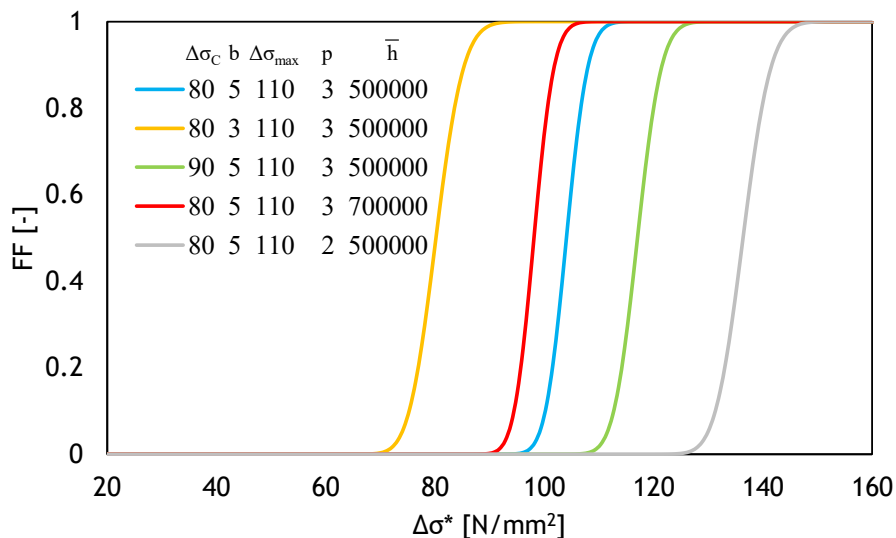


Figure 4.4-2 – Examples of FF curves derived according to the proposed approach.

#### 4.4.2 Corrosion fatigue fragility for steel bridges

In order to account for corrosion while addressing probabilistic PM of steel bridges, concepts discussed in Section 4.3 can be conveniently combined with assumptions for fatigue fragility, i.e., in order to estimate CFF in time. Namely (see also Figure 4.4-3):

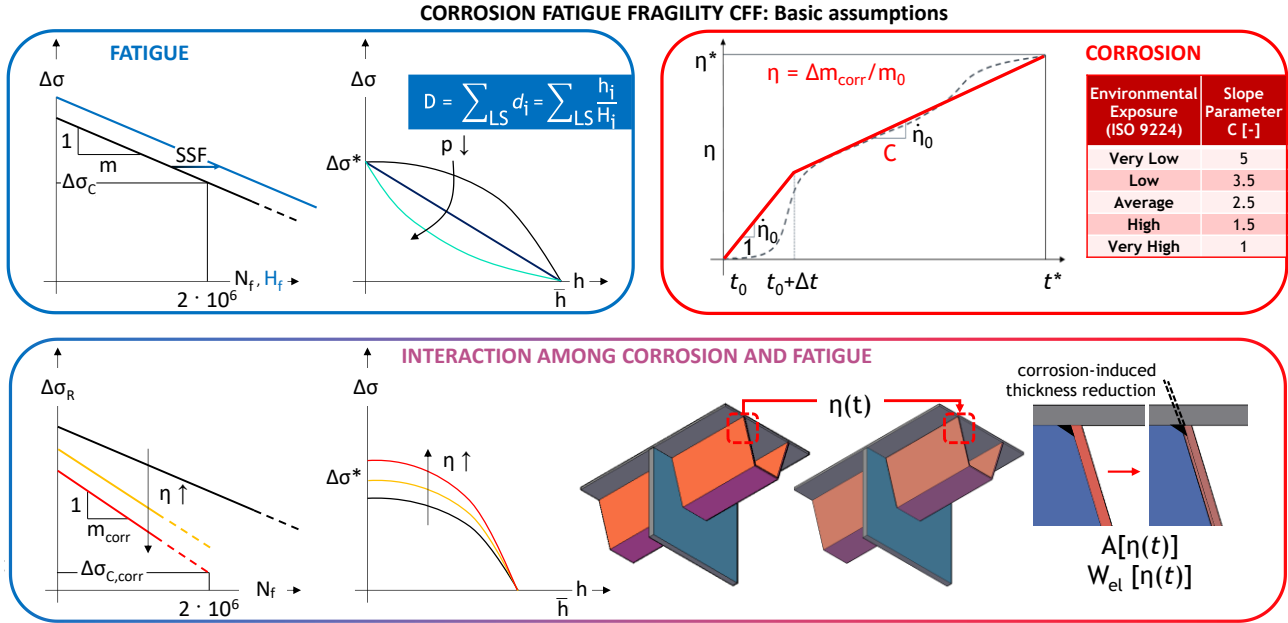


Figure 4.4-3 – Graphical representation of basic assumptions for the estimation of CFF according to the proposed approach.

- The corrosion-depending fatigue strength domain (S-N- $\eta$  curve) for a given detail can be expressed by introducing a dependance on corrosion degree  $\eta$  for Basquin parameters  $A(\eta)$ ,  $b(\eta)$ ] (Equation (23)):

$$N_f = A(\eta) \Delta\sigma^{-b(\eta)} \quad (23)$$

where  $A(\eta)$ ,  $b(\eta)$  can be derived according to Adasooriya et al. [19,21,45] (see Equations (11)-(16)). In presence of enough experimental data, corrosion dependence can be also introduced in the error parameter  $\varepsilon$ , i.e., by increasing its variance for increasing values of  $\eta$ . This expedient allows to account for non-uniform corrosion effect in terms of local stresses, as the “geometric” increase of stress due to the reduction of cross-section properties is instead included on the demand side (LS);

- Gassner curves for any given value of  $\eta$  can be still derived from Wholer ones by deriving the relevant SSF value;
- In order to account for overall cross-section loss, LS for the detail of concern can be upgraded as a function of  $\eta$ , i.e., by increasing the lone  $\Delta\sigma^*$  parameter and keeping  $\bar{h}$ ,  $p$  costant (Equation (24)):

$$\left( \frac{\Delta\sigma}{\Delta\sigma^*(\eta)} \right)^p = 1 - \frac{\log_{10} h}{\log_{10} \bar{h}} \quad (24)$$

where  $\Delta\sigma^*(\eta) = \Delta\sigma^*(\eta = 0) \cdot \frac{1}{1 - \frac{\log_{10} h}{\log_{10} \bar{h}}}$  according to the symbology from Equation (4), §4.3;

- Corrosive phenomena can be modelled as a progressive thickness decrease for details of interest, i.e., by assuming a time-depending  $\eta(t)$  trend complying with ISO 9224 prescriptions;

- Damage cumulation is now performed numerically as no closed-form solution can be derived. Namely, numerical integration to derive CFF can be done by treating infinitesimal damage  $dD$  as a tiny, yet finite, damage increment  $\Delta D_{i,i+1}$  defined as follows (Equation (25)):

$$\Delta D_{i,i+1} = \frac{k(t_i) [\Delta \sigma^*(\eta_i)]^{b(\eta_i)}}{SSF \cdot A(\eta_i)} \left\{ \left[ 1 - \frac{\log_{10} h_{i+1}}{\log_{10} \bar{h}} \right]^{b(\eta_i)/p} - \left[ 1 - \frac{\log_{10} h_i}{\log_{10} \bar{h}} \right]^{b(\eta_i)/p} \right\} \quad (25)$$

On the basis of the above assumptions, corrosion fatigue fragility CFF can be expressed via the following conditioned probability  $P(\text{Failure} \mid \text{Parameters})$  (Equation (26)):

$$CFF = P(D \geq D^* \mid \Delta \sigma^*, p, b, \bar{h}, t, \eta) \quad (26)$$

#### 4.4.3 Use of probabilistic formulations for predictive maintenance

Based on proposed developments, log-normal distributions for conditioned (corrosion) fatigue failure probability can be obtained for the CI of interest. Therefore, reliability index  $\beta$  can be immediately introduced by recalling its definition as the distance of the demand point  $(\Delta \sigma_c, m, \Delta \sigma^*, p, \bar{h}, \eta)$  from the distribution mean  $\mu$ , i.e., expressed as multiples of the distribution standard deviation  $\sigma$  (that is,  $CFF/FF(\Delta \sigma_c, m, \Delta \sigma^*, p, \bar{h}, \eta) = \mu - \beta\sigma$ ).

Therefore, reliability of existing bridges can be addressed by comparing the derived value of  $\beta$  with a minimum reliability threshold  $\beta_{\min}$  to be calibrated based on experimental evidences on real steel bridges (see Figure 4.4-4a). The relevant, optimal maintenance time  $\bar{t}$  can be hence estimated as the instant in which  $\beta(\bar{t}) = \beta_{\min}$ .

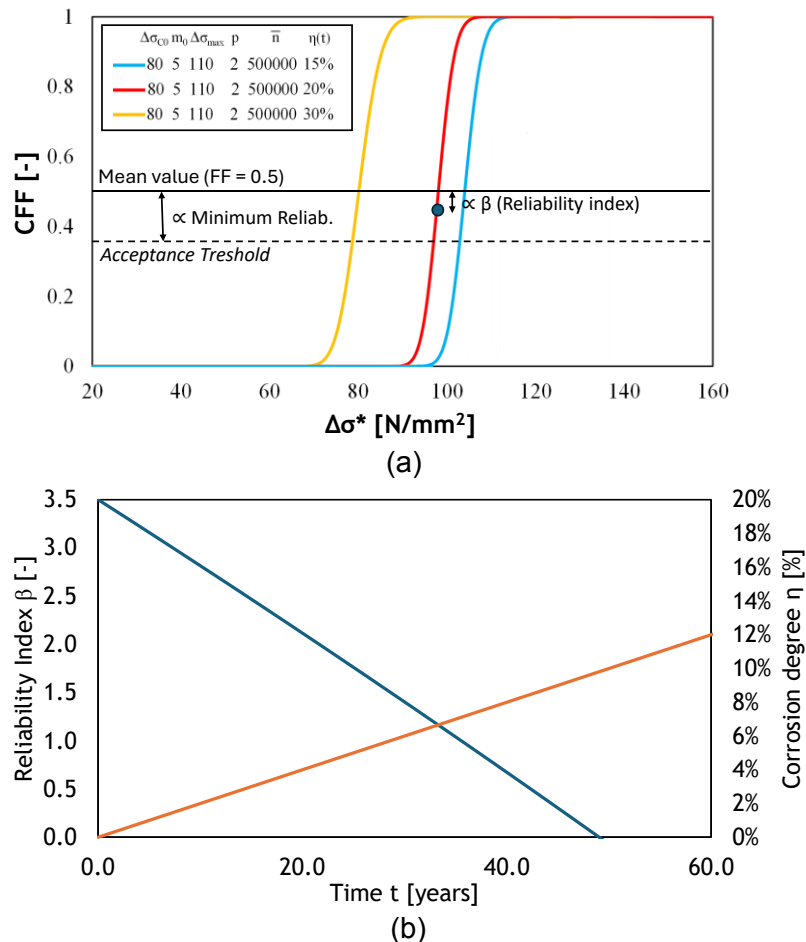


Figure 4.4-4 – (a) Graphical interpretation of reliability index for PM purposes, (b) Typical time-depending trend of corrosion degree and reliability index for a steel bridge.

## 5. Predictive Maintenance Methods for RC and PSC bridges

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### 5.1 Generality

The assessment of structural and seismic safety in existing reinforced concrete (RC) bridges is a critical topic in civil engineering. In Italy, a large portion of RC bridges was built before the implementation of modern standards for material durability and seismic design principles (e.g., [49-50]). Inadequate construction detailing, combined with exposure to aggressive environmental conditions, may lead to degradation processes, with steel reinforcement corrosion being one of the most frequent and critical.

Corrosion can occur uniformly, typically induced by humid and carbon dioxide-rich environments, or locally (referred to as “pitting” corrosion), due to the penetration of chloride ions, which is particularly common in bridges exposed to de-icing salts. The corrosion phenomenon leads to a reduction of the reinforcement bars’ cross-section and ductility capacity; moreover, it may cause cracking of the concrete cover and deterioration of the bond strength between steel and concrete [51-57].

From a durability perspective, the reinforcing bars corrosion phenomenon is commonly described through two main stages: (i) the “initiation” phase, during which aggressive agents infiltrate the concrete until the depassivation of steel reinforcements; and (ii) the “propagation” phase, during which mechanical degradation becomes increasingly significant. In the last decades, the mechanical behaviour of corroded RC elements has been extensively investigated through both experimental campaigns and numerical simulations (e.g., [51-53]), also addressing the effects of corrosion phenomena on material properties such as stress-strain relationships and bond degradation ([54-57]).

Clearly, the structural and seismic performance of RC bridge structures are strongly dependent on the mechanical properties of their components, thus corrosion-induced deterioration must be quantitatively included in both safety assessments and maintenance planning. Given the crucial role that bridges play in transportation networks and the potentially severe socio-economic impact of service disruptions due to retrofitting or replacement, several national and international guidelines have been developed to support the safety evaluation and maintenance of existing RC bridges (e.g., [58]). These guidelines typically adopt a tiered approach, involving visual inspections and “defect indices” to qualitatively classify damage severity and extent. However, visual inspection-based methods, while practical, would greatly benefit from integration with analytical and mechanics-based approaches capable of quantitatively linking observed damage to structural and seismic capacity reduction.

Therefore, in this Chapter, a framework for the safety evaluation and management planning of existing RC bridges is presented. In the proposed approach, the information obtained from visual inspections on observed defects to structural members is used to analytically evaluate their capacity reduction at the local level and a safety index (capacity-to-demand ratio) at the global level. Moreover, different corrosion propagation models are also employed to define upper and lower bounds for corrosion rates and assess safety reduction over time. Uncertainties related to degradation location and extent are also taken into account through a simplified probabilistic approach, and fragility relationships for degradation scenarios are derived.

Further details are provided in following subSections.

## 5.2 Framework for structural safety evaluation including degradation effects

The proposed methodology consists of a simplified yet mechanically-based procedure for the structural assessment of existing RC bridges affected by reinforcement corrosion. The framework aims to evaluate safety under either conventional traffic loads or seismic events, in line with performance-based seismic assessments and serviceability evaluations. The procedure is schematically illustrated in Figure 5.2-1.

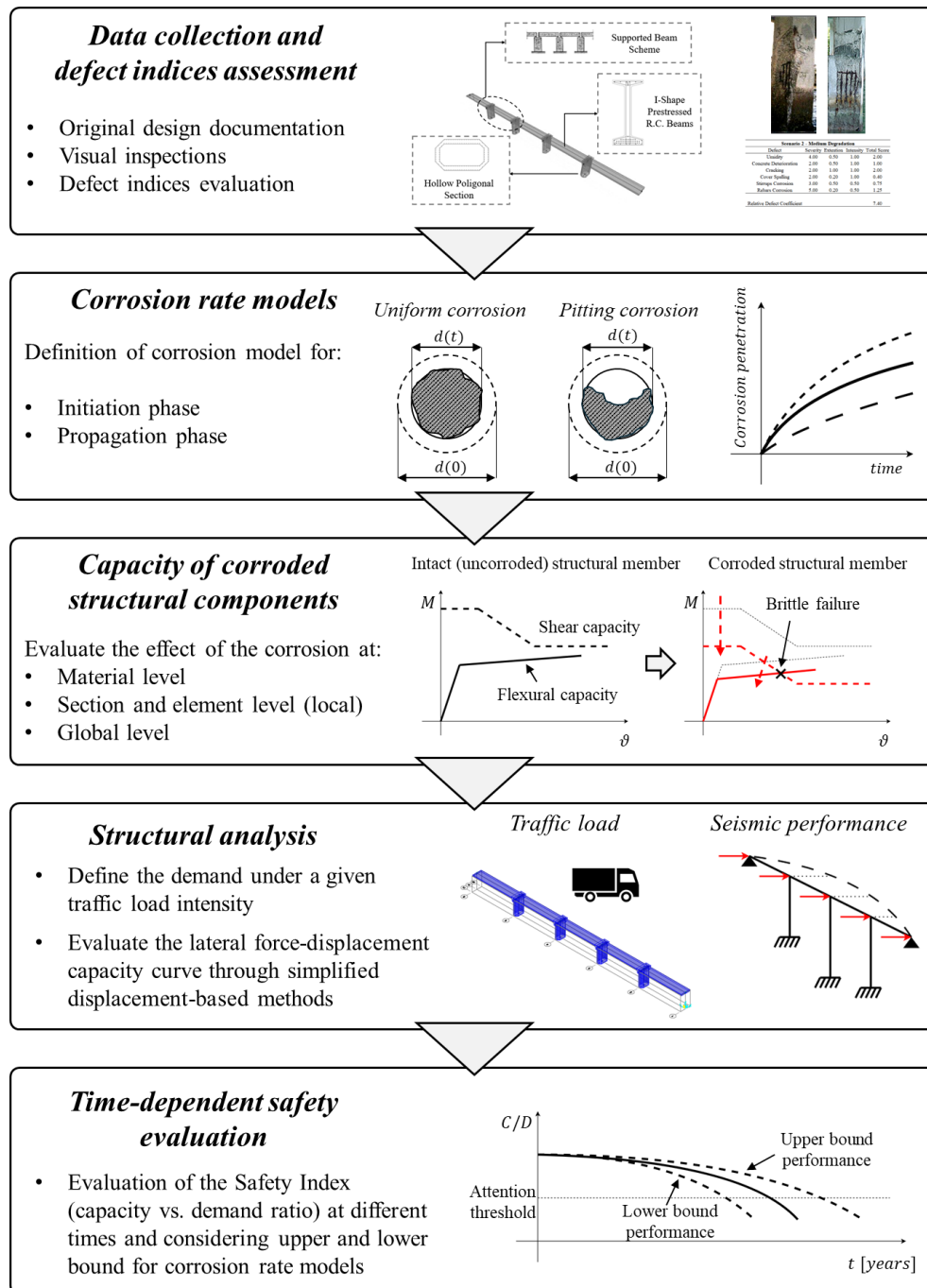


Figure 5.2-1 - Proposed framework for time-dependent safety evaluation of existing RC bridges. Each step of the procedure is described in the following subSections.

### 5.2.1 Data collection and defect indices assessment

The first fundamental step involves the systematic collection of available structural information through historical design documentation and in-situ observations via visual inspections. This is deemed a fundamental task to evaluate:

- Geometric characteristics of the bridge;
- Material mechanical properties;
- Structural and reinforcement details;
- Any defect affecting the structure.

Regarding the latter, in accordance with the *2020 Italian Guidelines on Risk Classification and Management of Bridges* [58], defect indices are assessed for different parts of the structure. These indices are derived from standardized defect forms, which allows for a classification of the severity and extent of various types of damage such as concrete cracking, spalling, humidity presence, or reinforcement corrosion. Focusing on RC and PSC bridges, the 2020 Italian Guidelines provide defect forms for the following structural components (depending on the bridge typology):

- Abutments;
- Piers;
- Bearings;
- Joints;
- Columns (vertical supports);
- Arches;
- Beams (either RC o PSC);
- Deck.

An example of a defect form for RC piers is reported in Figure 5.2-2. Moreover, the Italian Guidelines also provide ad-hoc defect reports to support the inspector in the identification of each defect and assess its severity, extension, and intensity (see Figure 5.2-3).

| Scheda Ispezione Ponti di Livello 1 |                                       |                               |                       |                          |                          |                          |                          |                          |                          |                          |    |                                |                          |                          |      |
|-------------------------------------|---------------------------------------|-------------------------------|-----------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|--------------------------|----|--------------------------------|--------------------------|--------------------------|------|
| 3                                   | Pile N _____                          | Strada di appartenenza: _____ | Progressiva km: _____ |                          |                          |                          |                          |                          |                          |                          |    |                                |                          |                          |      |
| Calcestruzzo                        |                                       | Tecnico rilevatore: _____     |                       |                          |                          |                          |                          |                          |                          |                          |    | Data ispezione: ____/____/____ |                          |                          |      |
| Codice difetto                      | Descrizione difetto                   | Visto                         | G                     | Estensione K1            |                          |                          | Intensità K2             |                          |                          | N° foto                  | PS | NA                             | NR                       | NP                       | Note |
|                                     |                                       |                               |                       | 0,2                      | 0,5                      | 1                        | 0,2                      | 0,5                      | 1                        |                          |    |                                |                          |                          |      |
| c.a./c.a.p._1                       | Macchie di umidità passiva            | <input type="checkbox"/>      | 1                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._2                       | Macchie di umidità attiva             | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Dif. Gen_1                          | Tracce di scolo                       | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._3                       | Cls dilavato / ammalorato             | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Dif. Gen_2                          | Ristagni d'acqua                      | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._4                       | Vespai                                | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._5                       | Distacco del copriferro               | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._6                       | Armatura ossidata/corrosa             | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._7                       | Lesioni a ragnatela modeste           | <input type="checkbox"/>      | 1                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._15                      | Lesioni in corrispondenza staffe      | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._8                       | Fessure orizzontali                   | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._9                       | Fessure verticali                     | <input type="checkbox"/>      | 2                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._10                      | Fessure diagonali                     | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._16                      | Staffe scoperte / ossidate            | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._11                      | Lesioni attacco pilastri              | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._23                      | Rottura staffe                        | <input type="checkbox"/>      | 4                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._17                      | Armatura longitudinale deformata      | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._12                      | Riprese successive deteriorate        | <input type="checkbox"/>      | 1                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Dif. Gen_3                          | Danni da urto                         | <input type="checkbox"/>      | 4                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Dif. Gen_6                          | Fuori piombo                          | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Ril/Fond_1                          | Scalzamento                           | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Ril/Fond_5                          | Movimenti di fondazione               | <input type="checkbox"/>      | 5                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| c.a./c.a.p._13                      | Lesioni da schiacciamento             | <input type="checkbox"/>      | 4                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Dif. Gen_4                          | Lesioni caratteristiche zone appoggio | <input type="checkbox"/>      | 3                     | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> | <input type="checkbox"/> |    | <input type="checkbox"/>       | <input type="checkbox"/> | <input type="checkbox"/> |      |
| Eventuali note                      |                                       |                               |                       |                          |                          |                          |                          |                          |                          |                          |    |                                |                          |                          |      |

Figure 5.2-2 – Defect form for RC piers (from [58]; in Italian).

| Rottura staffe                                                                                                                                                                                                                                                                  |       |                                                                            |              |       |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------|----------------------------------------------------------------------------|--------------|-------|
| <b>Peso del difetto</b>                                                                                                                                                                                                                                                         |       |                                                                            |              |       |
| G = 1                                                                                                                                                                                                                                                                           | G = 2 | G = 3                                                                      | <b>G = 4</b> | G = 5 |
| Estensione k <sub>1</sub>                                                                                                                                                                                                                                                       |       | 0,2 (appena presente)    0,5 ("50% superficie)    1 ("tutta la superficie) |              |       |
| Intensità k <sub>2</sub>                                                                                                                                                                                                                                                        |       | Sempre = 1                                                                 |              |       |
| <b>Descrizione</b>                                                                                                                                                                                                                                                              |       |                                                                            |              |       |
| <p>Si riferisce alla completa rottura delle armature trasversali degli elementi strutturali. Ciò spesso accade nelle travi di bordo dei ponti a travata, maggiormente esposte agli agenti esterni e quindi a fenomeni quali dilavamento e/o ammaloramento del calcestruzzo.</p> |       |                                                                            |              |       |
| <b>Cause</b>                                                                                                                                                                                                                                                                    |       |                                                                            |              |       |
| <p>L'esposizione continua delle staffe agli agenti esterni a seguito di distacchi di copriferro può provocare l'ossidazione e, nei casi più gravi, la riduzione della sezione resistente fino ad arrivare alla completa rottura.</p>                                            |       |                                                                            |              |       |
| <b>Fenomeni di degrado correlati</b>                                                                                                                                                                                                                                            |       |                                                                            |              |       |
| <p>Il fenomeno rappresenta l'evoluzione dell'<b>ossidazione delle staffe</b> in seguito a <b>distacchi di copriferro</b>. La presenza di staffe rotte può comportare la riduzione della capacità resistente degli elementi nei confronti delle sollecitazioni taglienti.</p>    |       |                                                                            |              |       |



Figure 5.2-3 – Example of defect record: failure of stirrups (from [58]; in Italian).

Each defect listed in the form is associated with a weight (level) of severity (G, where G=5 indicates the most severe level), extension (K<sub>1</sub>), and intensity (K<sub>2</sub>) coefficients. The inspector is also required to report:

- If the observed damage can compromise the structural stability of the bridge (PS; only for G=4 or 5);
- If the defect does not apply to the bridge typology under investigation (NA);
- If the defect can not be identified through visual inspection (e.g., NR);
- If the defect is not observed (NP)
- The numbering of photos for the specific defect; the latter must be adequately catalogued, numbered, and provided with captions clearly stating the defect typology and its location; a mapping of photographic viewpoints is required.

According to the Italian guidelines, based on the information collected through the visual inspection and the defect forms, it is also possible to define a Relative Defect Index (D<sub>r</sub>) for either each structural element, homogeneous groups of elements, or the entire bridge. Specifically, for each observed defect, the product  $G \cdot K_1 \cdot K_2$  represents a measure of the impact that the defect may have on maintenance strategies. The sum of these values for all observed defects ( $\sum_i G_i \cdot K_{1,i} \cdot K_{2,i}$ ) represents a measure of the overall condition of that element. This coefficient provides an initial qualitative indication of structural condition and is used to trigger the need for more detailed analyses. An example of possible degradation scenarios and related Relative Defect Index is shown in Figure 5.2-4. However, it is worth highlighting that, at the moment, these coefficients are not directly linked to a capacity reduction of the investigated RC structural members. Moreover, it should be noted that the severity of the defect is not dependent on its location. In many cases, this information can be crucial to evaluate if the observed defect can compromise the safety of the bridge.



| Scenario 1 - Low Degradation |          |           |           |             |
|------------------------------|----------|-----------|-----------|-------------|
| Defect                       | Severity | Extention | Intensity | Total Score |
| Umidity                      | 4.00     | 0.50      | 1.00      | 2.00        |
| Concrete Deterioration       | 2.00     | 0.50      | 1.00      | 1.00        |
| Cracking                     | 2.00     | 1.00      | 0.20      | 0.40        |
| Rebars Corrosion             | 5.00     | 0.20      | 0.20      | 0.20        |
| Relative Defect Coefficient  |          |           |           | 3.60        |

| Scenario 2 - Medium Degradation |          |           |           |             |
|---------------------------------|----------|-----------|-----------|-------------|
| Defect                          | Severity | Extention | Intensity | Total Score |
| Umidity                         | 4.00     | 0.50      | 1.00      | 2.00        |
| Concrete Deterioration          | 2.00     | 0.50      | 1.00      | 1.00        |
| Cracking                        | 2.00     | 1.00      | 1.00      | 2.00        |
| Cover Spalling                  | 2.00     | 0.20      | 1.00      | 0.40        |
| Stirrups Corrosion              | 3.00     | 0.50      | 0.50      | 0.75        |
| Rebars Corrosion                | 5.00     | 0.20      | 0.50      | 1.25        |
| Relative Defect Coefficient     |          |           |           | 7.40        |

Figure 5.2-4. Example of possible degradation scenarios and related Relative Defect Index (modified after [59])

### 5.2.2 Corrosion rate models

Once the passive protective layer of steel is compromised due to the entering of chlorides through the concrete cover, the reinforcement becomes vulnerable to corrosion. This marks the end of the initiation phase and the beginning of the active degradation process (i.e., “propagation” phase). In its early stages, corrosion typically manifests as isolated pits or micro-cracks randomly distributed along the rebar. With time, these discontinuities tend to expand and interconnect, progressively transforming into a more uniform pattern of attack often referred to as generalized corrosion (e.g., [56, 57]). In many cases, however, the deterioration does not proceed evenly. Instead, localized forms of corrosion develop, where the damage is concentrated in a few critical zones along the bar. This leads to the formation of deep pits, significantly reducing the local cross-section (“pitting” corrosion, Figure 5.2-5). Pitting corrosion typically results in more severe impacts on structural performance compared to uniform corrosion, particularly in terms of ductility and bond deterioration.

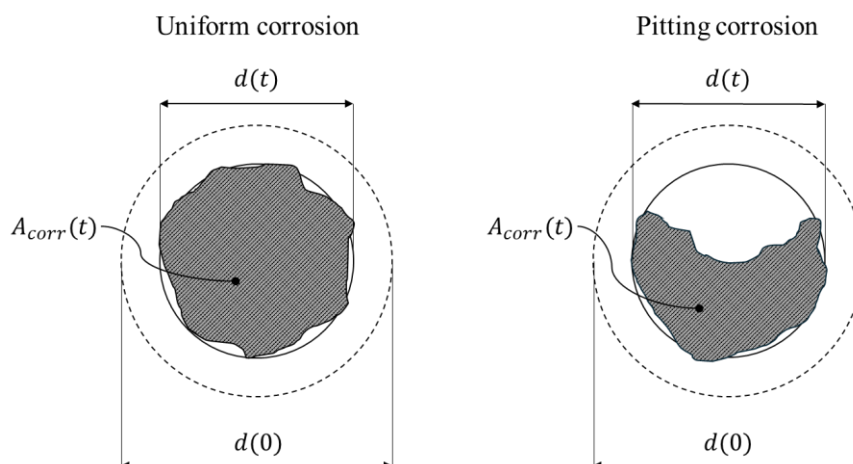


Figure 5.2-5. Schematic illustration of “uniform” and “pitting” corrosion.

One of the main objectives of the investigated predictive maintenance method is to account for time-dependent deterioration effects associated with corrosion in terms of structural safety. Concerning corrosion initiation, models based on Fick's second law (Equation (27)) can be used to evaluate chloride penetration into the concrete:

$$\frac{\partial C(x, t)}{\partial t} = - \frac{D_c \partial^2 C(x, t)}{\partial x^2} \quad (27)$$

where  $C(x, t)$  is the chloride concentration on the steels,  $x$  is chloride penetration depth,  $t$  is the time, and  $D_c$  is the diffusion coefficient. Chloride corrosion starts when the chloride content at the bars' depth exceeds a critical threshold, i.e.  $\partial C(x = c, t) = C_{cr}$  (where  $c$  is the concrete cover depth). As pointed out by Molaioni et al. [60], a common assumption consists of assuming that chloride content at the outer surface ( $C_s$ ) is constant, leading to the solution reported in Equation (28):

$$C(x = c, t) = C_s \left[ 1 - \operatorname{erf} \left( \frac{c}{2\sqrt{D_c t}} \right) \right] \quad (28)$$

where  $D_c$  is the diffusion coefficient, and  $\operatorname{erf}(\cdot)$  is the Gauss error function. As previously mentioned, the time to corrosion initiation,  $T_{corr}$ , can be evaluated as the time when the chloride concentration at the cover depth reaches the critical chloride concentration,  $C_{cr}$ . To this end, the probabilistic model proposed by DuraCrete [61] can be adopted, as reported in Equation (29) and Equation (30).

$$D_c(t) = k_e k_c k_t D_0 \left( \frac{t_0}{t} \right)^n \quad (29)$$

$$T_{corr} = X_1 \left\{ \frac{c^2}{4k_e k_c k_t D_0 t_0^n} \left[ \operatorname{erf}^{-1} \left( 1 - \frac{C_{cr}}{C_s} \right) \right]^{-2} \right\}^{\frac{1}{1-n}} \quad (30)$$

In in Equation (29) and Equation (30),  $D_0$  is the initial diffusion coefficient,  $t_0$  is the reference period, set equal to 28 days,  $n$  is the age factor,  $k_e$  is the environmental coefficient accounting for the temperature,  $k_c$  is the curing time coefficient,  $k_t$  is a correction factor accounting for the test method, and  $X_1$  is a parameter accounting for the model uncertainties related to the idealization of Fick's second law.

Concerning corrosion propagation, several corrosion rate models are available in the literature (e.g., [61-64]). An example of a comparison between different corrosion rate models is available in Gentile et al. [59] and is shown in Figure 5.2-6. As suggested by Gentile et al. [59], available models allow for the definition of lower and upper bounds of corrosion propagation rates. This enables the simplified assessment of structural safety degradation over time, for both seismic and traffic-related performance criteria. Yet, in the case of real applications, the information obtained from visual inspections and collected through defect form can be possibly used to estimate the most suitable corrosion rate models for a specific application. Moreover, the selection of the most suitable corrosion rate model over time (and the assessment of the related uncertainties) can be also supported by available monitoring data.

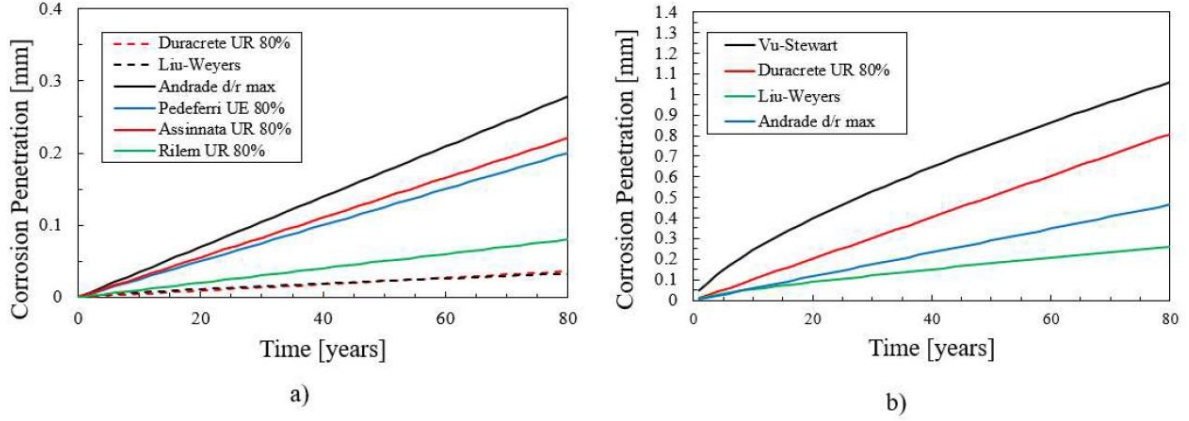


Figure 5.2-6 – Comparison between different corrosion rate models for a) carbonation and b) chlorides attack (after [59])

### 5.2.3 Corrosion effects on the capacity of RC structural members

Corrosion affecting RC structural members can compromise their stiffness, strength, and ductility capacity, potentially resulting in significant changes in the non-linear behavior. These effects may be related to multiple interrelated mechanisms, including: *i*) the reduction of the cross-sectional area of steel reinforcement; *ii*) a modification of the stress-strain response for steel material; *iii*) cracking in the concrete cover, due to the expansive action of corrosion; *iv*) degradation of the bond between steel and concrete; and *v*) deterioration of stirrups, leading to a loss of confinement in the concrete core.

To capture the effect of corrosion in the section analysis – through either simplified analytical calculation or a more-refined fiber numerical model approach - the nonlinear behavior of the construction materials is modified as a function of the rebars mass loss,  $M_{loss}$  [%], in terms of: *i*) steel reinforcement strength and deformation capacity; *ii*) concrete cover stiffness and strength; and *iii*) concrete core confinement.

The mechanical properties of reinforcement are modified based on the degradation laws proposed by Imperatore et al. [57], which were experimentally derived for both pitting and uniform corrosion. Equations (31)-(33) summarize the degradation laws related to uniform corrosion for yielding ( $f_y$ ) and ultimate ( $f_u$ ) strength and ultimate strain ( $\epsilon_{su}$ ):

$$f_y^{corr,unif} = f_y(1 - 0.0143453 \cdot M_{loss}[\%]) \quad (31)$$

$$f_u^{corr,unif} = f_u(1 - 0.0125301 \cdot M_{loss}[\%]) \quad (32)$$

$$\epsilon_{su}^{corr,unif} = \epsilon_s \exp(-0.0204983 \cdot M_{loss}[\%]) \quad (33)$$

Moreover, Equations (34)-(36) summarize the degradation laws related to pitting corrosion:

$$f_y^{corr,pitt} = f_y(1 - 0.019961 \cdot M_{loss}[\%]) \quad (34)$$

$$f_u^{\text{corr,pitt}} = f_u (1 - 0.018642 \cdot M_{\text{loss}}[\%]) \quad (35)$$

$$\varepsilon_{su}^{\text{corr,pitt}} = \varepsilon_s \exp(-0.0546993 \cdot M_{\text{loss}}[\%]) \quad (36)$$

The compressive strength of the concrete cover is also reduced to reflect cracking induced by corrosion by-products; to this end, the model proposed by Coronelli & Gambarova [65] can be adopted (Equation (37)):

$$f_c^{\text{corr}} = \frac{f_c}{1 + K\varepsilon_1/\varepsilon_{c0}} \quad (37)$$

where  $K$  is a coefficient related to bar diameter and roughness,  $\varepsilon_{c0}$  is the strain at the concrete peak compressive stress and  $\varepsilon_1$  is the “average (smeared) tensile strain in the cracked concrete at right angles to the direction of the applied compression” [65]. The value of  $\varepsilon_1$  can be estimated as a function of the  $M_{\text{loss}}[\%]$ . More details on the formulation can be found in the cited paper.

Finally, for concrete core, the confinement effect can be evaluated according to the model proposed by Mander et al. [66]. In this case, the effects of corrosion are accounted for by reducing the yielding strength of the stirrups. An example of the effect of corrosion on steel rebars, concrete cover and concrete core is shown in Figure 5.2-7.

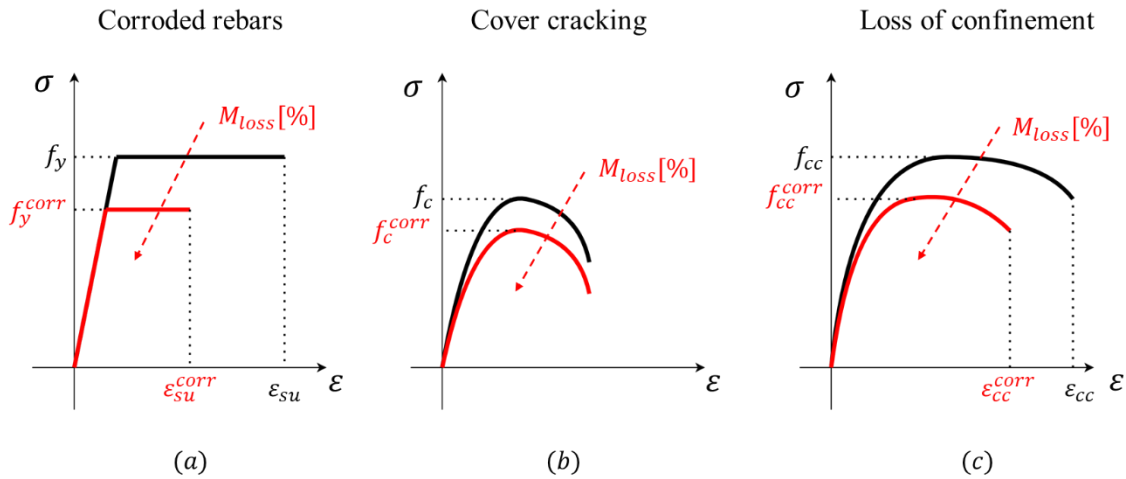


Figure 5.2-7. Qualitative example of the effects of corrosion on (a) steel rebars, (b) concrete cover and (c) concrete core.

The definition of stress-strain relationships of construction materials – either considering or neglecting the effects of corrosion – allows for the evaluation of the flexural and shear capacity at both the section and element levels through classical structural design principles. To evaluate the effect of corrosion on the strength and ductility capacity of the RC members, the moment-curvature relationships are evaluated, as well as the axial load–moment interaction diagrams. Moreover, the shear capacity is also evaluated according to the Italian Building Code [67,68] prescription. Possible flexural-shear interactions are also assessed. This is deemed a critical task, since the effects of

corrosion on stirrups may cause a change in the hierarchy of strengths at the local element scale, as shown in Figure 5.2-8.

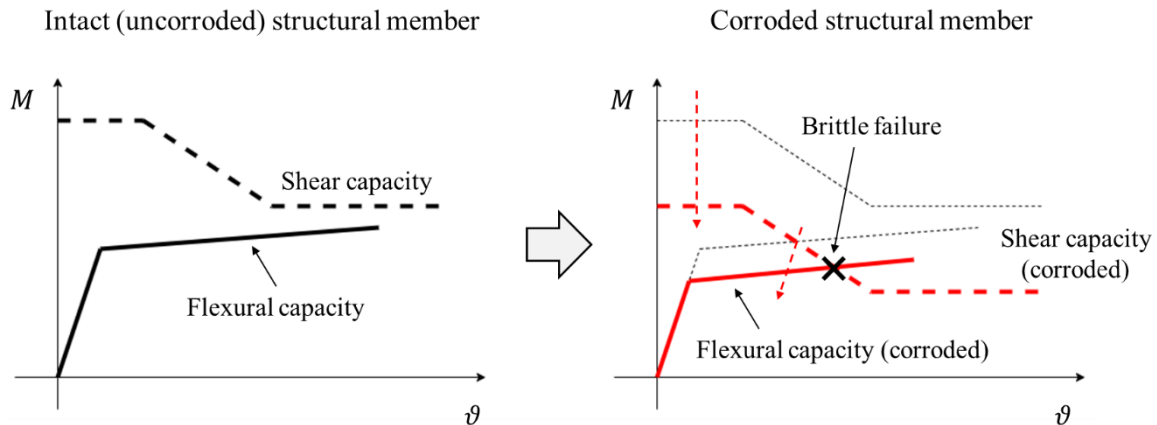


Figure 5.2-8. Example of modification of the hierarchy of strengths at the local component level due to the effects of corrosion.

An alternative approach for the evaluation of the flexural capacity of corroded RC structural members would consist of adopting capacity reduction factors for stiffness ( $\lambda_K$ ), strength ( $\lambda_Q$ ), and ductility ( $\lambda_D$ ) as a function of the level of corrosion. This methodology would lead to a common approach with the procedure proposed by the FEMA 306 report [69] to evaluate the seismic residual capacity of earthquake-damaged RC components. However, research effort is still needed to derive (either experimentally or numerically) these  $\lambda$ -factors for corroded components.

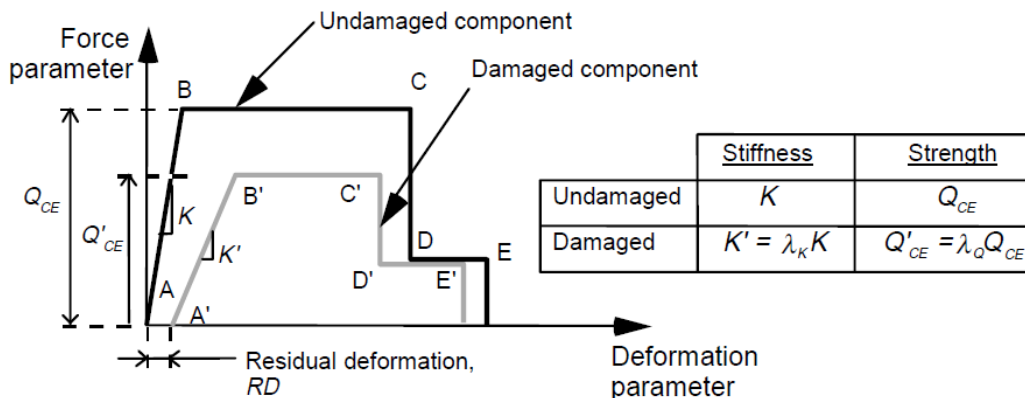


Figure 5.2-9. FEMA 306 approach for the evaluation of the seismic residual capacity of earthquake-damaged components: capacity reduction factors for stiffness, strength, and ductility (after [69]).

Finally, depending on the bridge typology and the level of refinement of the analysis, additional structural components can be also considered in the analysis. For instance, as suggested by Molaioni et al. [60], the effects of corrosion on bridge bearings can be considered according to the work of Mander et al. [70]. The latter includes a comprehensive comparison between experimental observations and analytical predictions across different types of steel bearings. This research work also provides practical recommendations for modelling approaches.

#### 5.2.4 Structural analysis

A structural analysis is needed to define the demand for each bridge's structural components under a given traffic load intensity. This task can be performed through either a numerical software-based simulation or available simplified procedures. The latter is deemed more suitable for developing a fast and updatable tool for predictive maintenance methods.

The first step of structural analysis consists of modelling the loads for the bridge deck, including: *i*) self-weight of deck components, *ii*) non-structural permanent loads, and *iii*) traffic loads. Specifically, traffic load can be assessed according to the load schemes defined by the Italian building code [67]. Clearly, the bridge typology and geometrical characteristics (e.g., number of lanes) strongly affect the evaluation of traffic loads. The tandem load position along the deck, as well as the scheme of loaded traffic lanes, should be defined to maximize the flexural and shear demand for the bridge's structural components.

Simplified structural analysis methods for RC e PSC bridges typically assume that the deck cross section can be considered transversely rigid (e.g., [71]). Another simplified hypothesis for prestressed concrete girder bridges is to neglect the torsional stiffness of the girders, which is deemed suitable for bridge superstructures characterized by a high length/width ratio (e.g., [72]). These simplified assumptions allow for the analytical calculation of the demand for bridge components, without the need for any numerical (software-based) simulation.

On the contrary, when dealing with seismic capacity, the global performance must be evaluated. The relationship between the local and global capacity is strictly dependent on the static scheme of the bridge. For instance, in simply supported systems, each pier may be analyzed independently, and, in a simplified approach, the overall bridge performance can be considered as governed by the most critical (i.e., weakest) element. In contrast, for continuous beam configurations, the global capacity in the longitudinal direction can be obtained by summing the capacities of individual piers assuming a uniform top displacement. In the transverse direction, a displacement-based method can be adopted, considering the modal contribution of each pier to global deformation [73], or adopting a static-based alternative [74].

#### 5.2.5 Time-dependent safety evaluation

For the practical application of the proposed maintenance framework, safety is firstly quantified by comparing the structural capacity with the demand. The resulting Safety Index (i.e., Capacity/Demand ratio) is deemed a useful indicator to guide decisions regarding retrofitting, maintenance, or operational limitations.

When dealing with traffic loads, as mentioned in the previous section, the demand is assessed considering the design load models provided by the Italian Building Code ([67]), and compared with the capacity (strength) of the structural components considering also the effects of corrosion. Differently, when dealing with seismic safety, the capacity vs. demand comparison is performed in the Acceleration Displacement Response Spectrum (ADRS), in line with the Capacity Spectrum Method [75], or the N2 method [76].

By carrying out safety evaluations at different times of the bridge's life, it is possible to estimate the decay of the Safety Risk index over time. To this end, the corrosion rate models described in Section 5.2.2 can be adopted to estimate the reduction in terms of structural capacity due to degradation effects, as described in Section 5.2.3. This enables the assessment of structural safety degradation over time, for both seismic and traffic-related performance. Moreover, if suitable traffic data are

available, the predictive maintenance method can also account for the expected increase in traffic demand over time (e.g., [77]).

An example of time-dependent seismic safety evaluation through the CSM is shown in Figure 5.2-10.

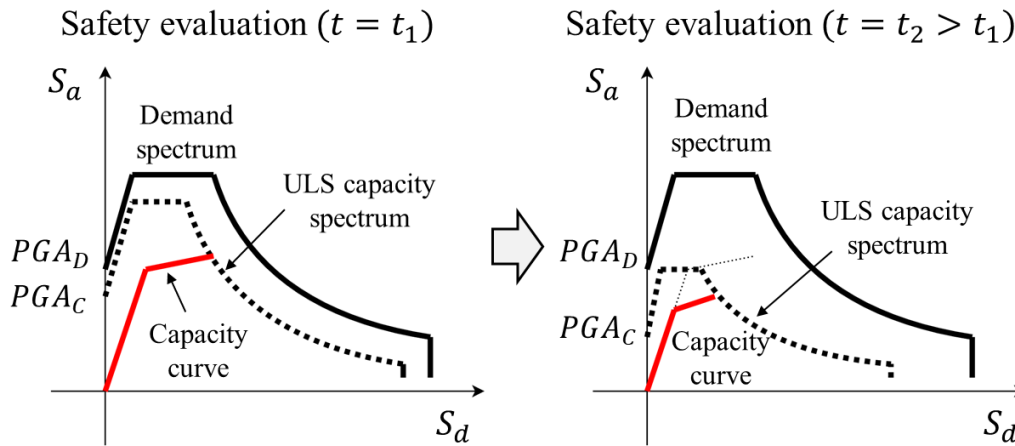


Figure 5.2-10 – Example of time-dependent performance points evaluation at Ultimate Limit State (ULS) in the ADRS domain through the Capacity Spectrum Method.

Considering an upper and lower bound corrosion rate model, the reduction in terms of Safety Risk index over time can be evaluated. As an example, Figure 5.2-11a shows a possible definition of corrosion intensity evolution for different scenarios (see Figure 5.2-4 for the scenario definition). These models allow for the evaluation of the reduction in terms of safety index over time, as shown in Figure 5.2-11b.

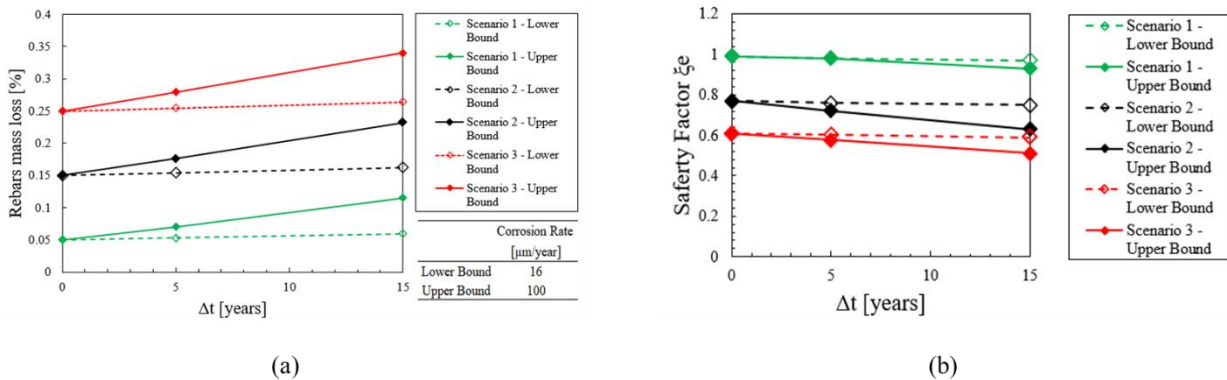


Figure 5.2-11 – Examples of (a) and (b) safety index reduction over time due to corrosion development (after [59]).

Alternatively, a probabilistic-based approach can be adopted. This method required the definition of the uncertainties related to the structural capacity, the effects of corrosion, and the demand – either traffic and/or seismic - (e.g., [60,71,72,77]). Regarding corrosion effects, it is important to note that visual inspections and defect index-based approaches do not provide direct relationships between observed damage and mechanical capacity reduction, and uncertainty related to the location of the critical section, the amount of lost mass and the number of bars involved should be considered in the analysis.

In the probabilistic approach, fragility models are developed for either traffic load (e.g., [71,72]) or seismic actions (e.g., [59,60]). For instance, in the case of seismic assessment, time-dependent

fragility relationships define the probability of failure (i.e., reaching or exceeding a specific damage state),  $P(f|IM, t)$ , given an intensity measure  $IM$  and evaluated at a specific time  $t$  (Figure 5.2-12).

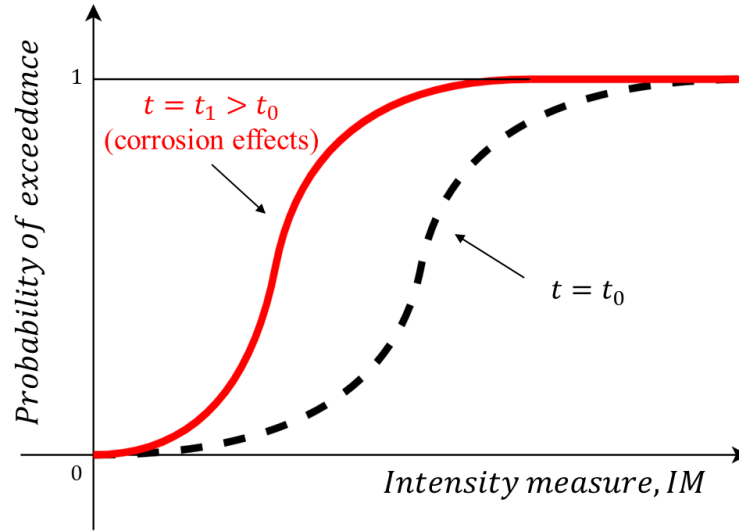


Figure 5.2-12 - Example of time-dependent fragility model.

By integrating the fragility with a probabilistic model for the demand (e.g., for seismic demand, the seismic hazard curve of the site,  $\lambda_{IM}$ , obtained through a Probabilistic Seismic Hazard Analysis, PSHA), it is possible to evaluate the mean annual rate of failure  $\lambda_f$  for the investigated bridge. To account for the time-dependent effects of corrosion on the structure, this evaluation should be performed at different time steps, as reported in Equation (38):

$$\lambda_f(t) = \int_0^{\infty} P(f|im, t) \cdot |d\lambda_{IM}(im)| \quad (38)$$

Note that in Equation (38) it is assumed that the seismic hazard curve  $\lambda_{IM}$  is not dependent on time. The  $\lambda_f(t)$  can be used to evaluate the probability of failure in the useful life of the bridge  $P_{f,T}(t)$ , assuming that the occurrence of earthquakes follows a Poisson process (Equation (39)):

$$P_{f,T}(t) = 1 - e^{-\lambda_f(t)T} \quad (39)$$

Finally, the reliability index  $\beta$  can be computed at different times through Equation (40):

$$\beta(t) = -\Phi^{-1}(P_{f,T}(t)) \quad (40)$$

Where  $\Phi$  is the standard normal cumulative distribution function. The final output of the probabilistic procedure consists of an evaluation of the reduction in terms of reliability index over time, in analogy with Figure 5.2-11b. This result can be compared with target values of the reliability index, defined according to the main national and international codes and guidelines.

## 6. Conclusions

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In the present DV, both data-based and analytic predictive maintenance methods (PMMs) have been thoroughly discussed with reference to steel, RC, PSC and composite bridges. On the basis of illustrated developments, the following conclusive remarks can be pointed out:

- The structural performance of critical bridge infrastructures in service is strongly affected by environmental and man-induced degradation phenomena, with most relevant problems being *i*) corrosion of members (steel & composite CIs) and components such as rebars and pre-stressing tendons (RC and PSC CIs) and *ii*) fatigue (steel & composite CIs);
- In order to avoid unbearable economic and social impacts, time-dependent performance of such CIs should be properly monitored – and, when relevant and possible, extended – with the aid of consistent PMMs;
- In this DV, both data-based and analytic PMMs for steel, RC, PSC and composite bridges were discussed;
- For critical bridge infrastructures, structural “augmentation” (i.e., service life extension) can be suitably achieved via structural health monitoring, i.e., enabling a data-based and time-evolving assessment of the CI of concern which can support decision making for the bridge owner/manager;
- Alternatively, analytical PMMs can be used for critical bridge structures. With reference to steel & composite ones, a first approximated and deterministic technique and a more robust, probabilistic approach were presented to address corrosion fatigue issues and derive the most convenient maintenance time according to reliability considerations;
- In a similar fashion, analytical PMMs for RC/PSC bridges were hence discussed, i.e., aiming at performing a time-dependent safety assessment while accounting for increasing corrosion in rebars. To this end, the evolution of degradation phenomena and their impact on the static performance of critical concrete members was addressed and framed within both *i*) a N2-based check format and *ii*) a time-dependant fragility (i.e., probabilistic) format.

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multi-Risk sciEnce for resilienT commUnities undeR a changiNg climate

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## 1. Technical references

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## 2. ABSTRACT

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Work Package 6 (WP6) addresses integrated solutions for holistic risk reduction in Critical Infrastructures (CIs). Transportation, hydraulic and energy/telecommunication infrastructures are treated accounting for their different peculiarities. To this end, effective and efficient maintenance strategies play a crucial role among research activities carried out within WP6.

In this context, predictive maintenance methods (PMMs) are addressed in detail in Task 6.1, i.e., with the aim of developing an IT framework for the on-site implementation of such methods.

DV6.1b represents the second Section of a two-parts document regarding research advances on PMMs applied to CIs. In particular, software tools for PM of steel, composite and RC bridges will be presented and relevant architecture details will be thoroughly discussed. Some relevant applicative examples will be also reported.

Developed software is made available through the RETURN Foundation for research purposes.

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## 4. Introduction

### 4.1 Background and motivation

Critical infrastructures, e.g. transport, industrial and power grid infrastructures are essential for ensuring the safety and socio-economic growth of communities. Nevertheless, existing CIs are often outdated and vulnerable. In past times, most efforts from research and technical practice were devoted to finding suitable retrofit interventions for maintenance of damaged CIs. However, this approach proved to be occasionally ineffective as structural upgrading was only performed after damaging events (e.g., aftermaths from long-term ground motions – slow landslides, subsidence – or from extreme events such as strong earthquakes). As a result, severe losses in terms of repairing costs and temporary inoperability were sometimes achieved.

In response to such criticalities, predictive maintenance methods (PMMs) recently emerged as an efficient and effective alternative to after-damage repairing. Namely, on the basis of suitable monitoring data (i.e., through the help of dedicated SHM systems), the ideal maintenance time for CIs can be derived to balance costs and outcomes within a reliability framework (see Figure 4.1-1).

In this regard, different aspects concerning PMMs are still under investigation nowadays. For instance, while consolidated techniques are available for practitioners addressing building structures, very few indications are available in literature and/or in normative provisions with respect to linear infrastructures such as bridges. Moreover, some complex aspects such as the influence of degradation phenomena on the static and cyclic performance of CIs still need further inquiry.

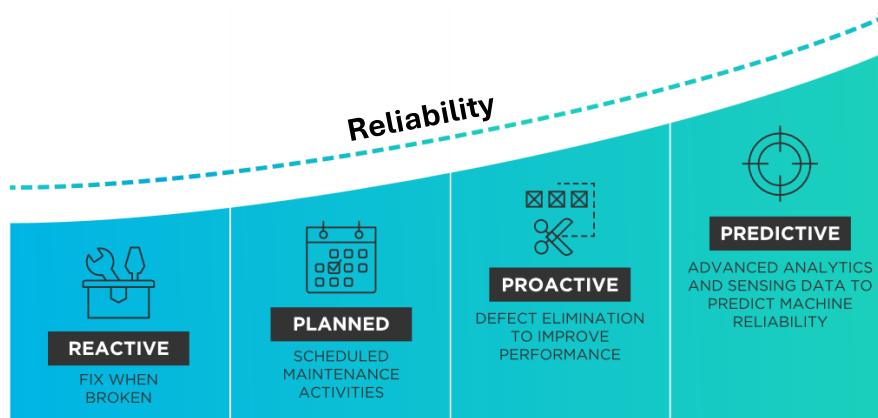


Figure 4.1-1 – Maintenance methods for Critical Infrastructures and relevant reliability level.

### 4.2 Outline of the report

The present report is divided into two main parts. Namely, software tools for PM of steel and composite bridge infrastructures are addressed in Section 5 with the aid of a relevant example drawn for POC2 (Cadore Bridge). Subsequently, software tools for PM of RC/PSC bridges are discussed in Section 6. Finally, main conclusive remarks are summarized in Section 8. Each of the following Sections is articulated in relevant subSections to address the following key aspects: 1) Underlying methodology (recall on main aspects discussed in DV6.1a), 2) Software details – architecture, algorithms, etc..., 3) Application through example(s).

## 5. Software tools for PM of Steel/Composite Bridges

### 5.1 Underlying methodology for probability-based PM

As deeply discussed in DV6.1a, addressing PM of steel/composite bridges in a probabilistic framework through analytical formulations is a complex task.

In order to do so, some simplifying assumptions were hence introduced in DV6.1a, i.e., in order to derive a corrosion fatigue fragility (CFF) curve for CIs. This enables the estimation of the most suitable maintenance time  $\bar{t}$  via the well-known reliability index  $\beta$  as defined in EN1990. Adopting such analytical approach allows for a convenient computational implementation of required calculations.

Before discussing the relevant software tool features (see §5.2), key aspects of the proposed methodology are first briefly reintroduced (the interested reader may refer to DV6.1a for further details). Subsequently, a relevant application is presented with reference to POC2 Cadore Bridge (see §5.4).

#### 5.1.1 Recall of the proposed methodology: Fatigue fragility for steel bridges

The proposed method for analytic derivation of FF in absence of corrosive phenomena is based on few simplified assumptions, i.e. (see Figure 5.1-1):

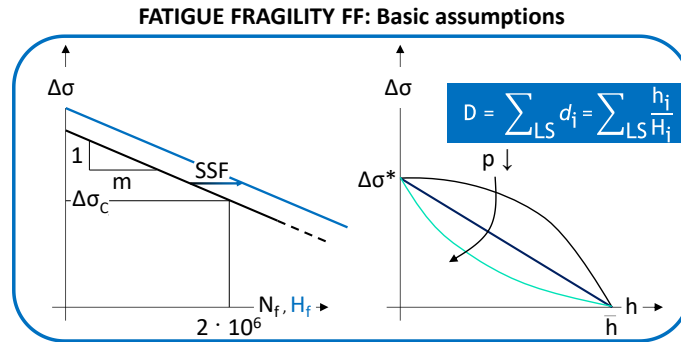


Figure 5.1-1 – Graphical representation of basic assumptions for the estimation of FF according to the proposed approach.

- The fatigue strength domain (S-N curve) for a given detail can be expressed via a 2-parameter (A, b) Basquin's Law (Equation (1)):

$$N_f = A \Delta \sigma^{-b} \quad (1)$$

For a given detail, S-N curves for a different confidence interval are parallel to each other. Randomness of the S-N curve can be hence expressed through a single error parameter  $\varepsilon$  with null expected value and variance estimated based on experimental data;

- Fatigue strength for variable-amplitude loads can be analytically derived from the Wohler curve, i.e., provided that it is expressed with respect to cumulated cycles  $H_f = \sum_i N_i$  via the so-called spectrum shape factor SSF as defined by Gassner (Equation (2)):

$$\begin{cases} H_f = \text{SSF} \cdot A(P_F) \Delta \sigma^{-b} \\ H_f = \sum_i N_i \end{cases} \quad (2)$$

where  $SSF \geq 1$  accounts for the load spectrum (LS) shape (i.e.,  $SSF = 1$  is achieved only for a constant-amplitude fatigue loading). Typical values of  $SSF$  for steel bridges are in the range  $2 \div 3$ , with the Gassner curve slope being still equal to  $b$ .

- LS for the detail of concern can be expressed through a 3-parameter ( $\Delta\sigma^*$ ,  $\bar{h}$ ,  $p$ ) expression as follows (Equation (3)):

$$\left(\frac{\Delta\sigma}{\Delta\sigma^*}\right)^p = 1 - \frac{\log_{10} h}{\log_{10} \bar{h}} \quad (3)$$

where  $h = \sum_i n_i$ ,  $\Delta\sigma^*$ ,  $\bar{h}$  are reference  $\Delta\sigma$ ,  $h$  values, and  $p$  governs the LS shape;

- Damage cumulation due to variable-amplitude fatigue can be addressed by extending Miner's rule to the cumulated cycles' domain  $H_f$ -N.

On the basis of the above assumptions, fatigue fragility FF can be expressed via the following conditioned probability  $P(\text{Failure} \mid \text{Parameters})$  (Equation (4)):

$$FF = P(D \geq D^* \mid \Delta\sigma^*, p, b, \bar{h}, t) \quad (4)$$

where  $D$  is the total fatigue damage estimated through integration of infinitesimal damage over the LS (continuous Miner's rule), while  $D^*$  is the damage threshold for which fatigue failure is assumed. As reported in Zhao (1995),  $D^*$  can be assumed as a random variable with unitary mean and 30% variance.

Moreover, an explicit dependence on time  $t$  is introduced to estimate FF at any given time during service life, i.e., through a normalized coefficient  $k(t)$ . By replacing Equations (1)-(3) into Equation (4), the following expression for FF can be derived (Equation (5)):

$$FF = \Phi \left\{ \frac{1}{s} \log \left[ C^* \frac{k(t)}{A} \left( \frac{\Delta\sigma^*}{\log h^{1/p}} \right)^b \bar{h} \right] \right\} \quad (5)$$

where  $s$  accounts for the combined standard deviation of  $H_f$ -N curve and damage threshold  $D^*$ ,  $k(t)$  is defined as the ratio among the journeyed traffic JT in the period  $t_0 \div t$  and the total traffic TT in the entire analysis window and  $C^*$  is a convenient constant depending on  $p$ ,  $b$ .

$C^*$  values for most common values of  $p$ ,  $b$  are provided in Table 5.1-1.

Table 5.1-1 – Values of constant  $C^*$  to estimate FF for typical values of  $p$ ,  $b$ .

| LS shape parameter $p$ [-] | Fatigue domain slope $b$ [-] | Constant $C^*$ [-] |
|----------------------------|------------------------------|--------------------|
| 1.5                        | 3.0                          | 2.1                |
| 1.5                        | 5.0                          | 9.7                |
| 2.0                        | 3.0                          | 1.4                |
| 2.0                        | 5.0                          | 3.5                |
| 3.0                        | 3.0                          | 1.0                |
| 3.0                        | 5.0                          | 1.6                |

### 5.1.2 Corrosion fatigue fragility for steel bridges

In order to account for corrosion while addressing probabilistic PM of steel bridges, concepts discussed in Section 5.1.1 should be properly integrated with *i*) a consistent corrosion development model and *ii*) a corrosion-depending fatigue strength domain, i.e., in order to estimate corrosion fatigue fragility CFF in time. Namely (see also Figure 5.1-2):

CORROSION FATIGUE FRAGILITY CFF: Basic assumptions

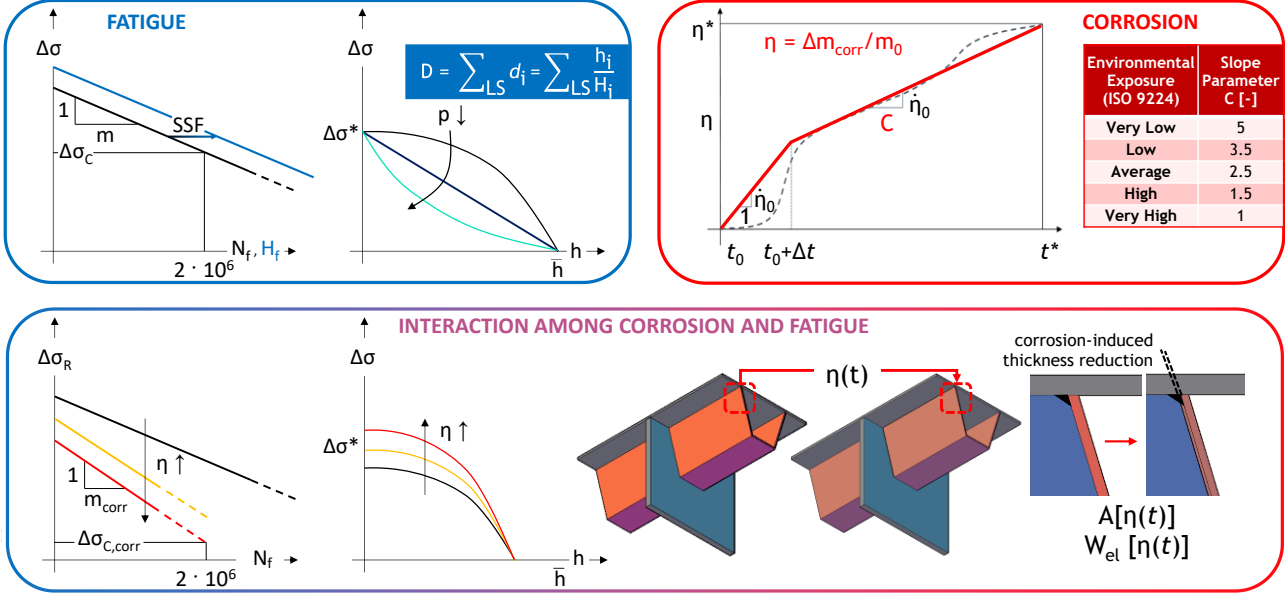


Figure 5.1-2 – Graphical representation of basic assumptions for the estimation of CFF according to the proposed approach.

- The corrosion-depending fatigue strength domain (S-N- $\eta$  curve) for a given detail can be expressed by introducing a dependance on corrosion degree  $\eta$  for Basquin parameters  $A(\eta)$ ,  $b(\eta)$ ] (Equation (6)):

$$N_f = A(\eta) \Delta\sigma^{-b(\eta)} \quad (6)$$

where  $A(\eta)$ ,  $b(\eta)$  can be derived according to *Adasooriya et al. (2019)* (Equations (7)-(8)):

$$\log \frac{5 \times 10^6}{N_i} = b_1(\eta) \log \left( \frac{\Delta\sigma_{D,\eta}}{\Delta\sigma_{Eq,i}} \right) \quad (7)$$

$$\log \frac{N_i}{5 \times 10^6} = b_2(\eta) \log \left( \frac{\Delta\sigma_{Eq,i}}{\Delta\sigma_{D,\eta}} \right) \quad (8)$$

with  $\Delta\sigma_{D,\eta}$  being the modified constant amplitude fatigue limit (CAFL) for the structural detail of interest, accounting for the effect of corrosion (i.e., the stress range which induces corrosion fatigue failure for  $N = 5 \times 10^6$ ),  $b_1(\eta)/b_2(\eta)$  being the corrosion-depending inverse slopes of the LCF and HCF branch of the S-N curve, respectively and  $A(\eta)$  can be derived by grouping all constant terms.

According to *Adasooriya et al. (2019)*, the modified CAFL can be derived assuming that very low-cycle fatigue (VLCF) behavior is unaffected by corrosion, i.e., the stress range  $\Delta\sigma_{10000}$  inducing fatigue collapse for  $N = 10^4$  does not depend on  $\eta$ . Thus,  $\Delta\sigma_{D,\eta}$  can be calculated via Equation (9):

$$\Delta\sigma_{D,\eta} = \frac{\Delta\sigma_{10000}}{(5 \times 10^2)^{\frac{1}{b_{1,\eta}}}} \quad (9)$$

$\Delta\sigma_{10000}$  is immediately derived from the detail class  $\Delta\sigma_c$ , i.e., the stress range causing fatigue collapse for  $N = 2 \times 10^6$  in the absence of corrosion (Equation (10)):

$$\Delta\sigma_{D,10000} = (2 \times 10^2)^{\frac{1}{b_{1,0}}} \cdot \Delta\sigma_c \quad (10)$$

For the sake of clarity,  $b_{1,0}$  is the inverse slope of the LCF branch according to EN1993:1–9 in pristine conditions. Consistently with the results reported in *Adasooriya et. al (2019)*, the reciprocal slopes are assumed to linearly reduce as  $\eta$  increases (Equations (11)–(12)):

$$b_{1,\eta} = b_{1,0} - \frac{\eta}{\eta_{ref}} \Delta b_{1,\eta} \quad (11)$$

$$b_{2,\eta} = b_{2,0} - \frac{\eta}{\eta_{ref}} \Delta b_{2,\eta} \quad (12)$$

in which  $b_{1,0}/b_{2,0}$  are the inverse slopes of the S-N curve for LCF/HCF in pristine conditions, respectively;  $b_{1,\eta}/b_{2,\eta}$  are the corrosion-affected inverse slopes for the same branches; and  $\eta_{ref}/\Delta b_{1,\eta}/\Delta b_{2,\eta}$  are the experimental parameters expressing the influence of corrosion on the fatigue behavior of the steel components.

According to indications reported in *Milone & Landolfo (2022)*,  $\eta_{ref} = 0.2$ ,  $\Delta b_{1,\eta} = 0.375 b_{1,0}$ , and  $\Delta b_{2,\eta} = 0.375 b_{2,0}$  are assumed. In this way, the observed increase of the slopes (i.e.,  $\approx 1.6$  times their original values) is attained for the mean value of corrosion detected in the reference experimental tests (i.e.,  $\eta_{mean} \approx 20\%$ ).

Conservatively, and in line with the experimental evidence, for the corrosion fatigue domain no endurance limit is assumed, i.e.,  $b_{2,\eta} < \infty$ . The resulting shape of the S-N- $\eta$  domains, according to the above Equations, is depicted in Figure 5.1-3 for increasing  $\eta$ .

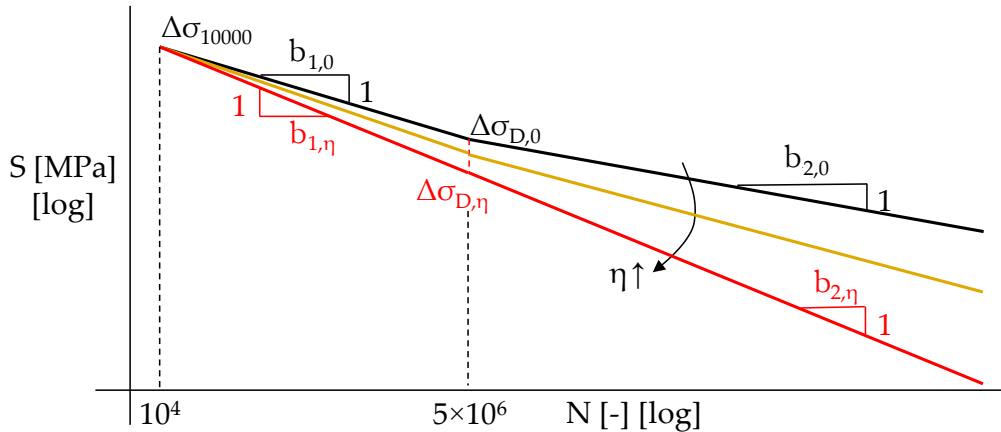


Figure 5.1-3 – Assumed shape of S-M- $\eta$  domains for corrosion fatigue analyses.

In presence of enough experimental data, corrosion dependence can be also accounted by increasing the variance  $s^2$  of the error parameter  $\varepsilon \sim \text{LN}(0,s)$  for increasing values of  $\eta$ . In the following, the simplified assumption of  $s^2$  being constant and equal to the relevant value for pristine specimens will be instead assumed;

- Gassner curves for any given value of  $\eta$  can be still derived from Wohler ones by deriving the relevant SSF value;
- In order to account for overall cross-section loss, LS for the detail of concern can be upgraded as a function of  $\eta$ , i.e., by increasing the lone  $\Delta\sigma^*$  parameter and keeping  $\bar{h}$ ,  $\rho$  constant (Equation (13)):

$$\left( \frac{\Delta\sigma}{\Delta\sigma^*(\eta)} \right)^p = 1 - \frac{\log_{10} h}{\log_{10} \bar{h}} \quad (13)$$

In which  $\Delta\sigma^*(\eta) = \sum_i k_i(\eta) \times \Delta\sigma_i^*(\eta = 0)$  with the following meaning of symbols:

- $k_i(\eta)$  is the  $i$ -th corrosion-dependent stress amplification factor related to the reduction of the  $i$ -th geometrical property of interest for elastic calculations (e.g., area  $A$  for axial force-induced normal stress range  $\rightarrow k_A$ , elastic modulus  $W$  for bending moment-induced normal stress range  $\rightarrow k_W$ , etc...);
- $\Delta\sigma_i^*(\eta = 0)$  is the maximum stress range value in pristine conditions (null  $\eta$ ) related to the  $i$ -th internal action of concern (axial force, bending moment, etc...).
- Corrosive phenomena can be modelled as a progressive thickness decrease for details of interest, i.e., by assuming a time-dependent  $\eta(t)$  trend complying with ISO 9224 prescriptions (Figure 5.1-4, red solid curve):

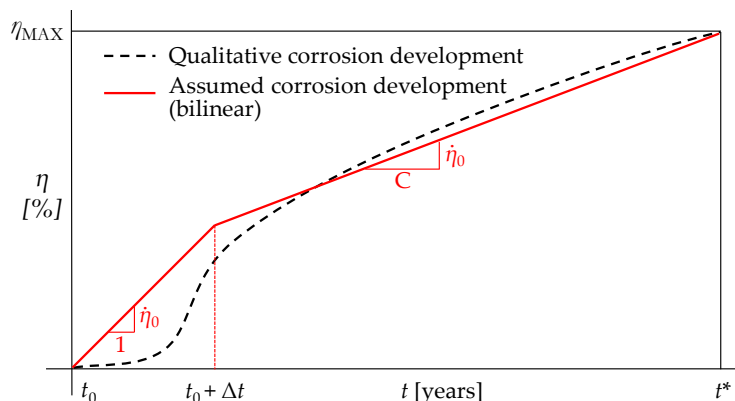


Figure 5.1-4 – Qualitative (black dashed curve) and assumed (red solid polyline) trends of corrosion development for the proposed PMM.

Therefore, according to prescriptions from ISO 9224, corrosion development is approximated by a polyline with two branches (i.e., with a piecewise constant corrosion rate).

The ratio among the slopes of the consecutive branches is equal to  $C : 1$ , with  $C \geq 1$  depending on the corrosivity category of concern. The suggested values of parameter  $C$  for each category are summarized in Table 5.1-2.

Table 5.1-2 – Suggested values of  $C$  for each corrosivity category according to ISO 9224.

| Corrosivity Category | Exposition | $C$ [-] |
|----------------------|------------|---------|
| C1                   | Very low   | 5       |
| C2                   | Low        | 3.5     |
| C3                   | Medium     | 2.5     |
| C4                   | High       | 1.5     |
| C5                   | Very high  | 1       |

The knee point of the bi-linear curve is assumed to occur  $\Delta t$  years after construction time  $t_0$ . According to the recommendations from ISO 9224, in absence of more precise information,  $\Delta t = 10$  years is assumed. The corrosion degree  $\eta$  is assumed null for  $t = t_0$ . Conversely, the maximum corrosion degree  $\eta_{MAX}$  is assumed to occur in correspondence with the target fatigue life  $t^*$ . In absence of more detailed indications,  $t^*$  can be assumed as coincident with the reference life  $L_{REF}$  of the CI of interest ( $\approx 100$  years for a steel/composite bridge).

Consistently with the assumption of a piecewise constant corrosion rate, the corrosion degree at a given time  $\eta(t)$  within the service life is expressed by Equations (14)-(15):

$$\eta(t) = \begin{cases} \eta_0 t & t \leq \Delta t \\ \frac{\eta_0}{C} t + \Delta t \eta_0 \left(1 - \frac{1}{C}\right) & \Delta t < t \leq t^* \end{cases} \quad (14)$$

$$\eta_0 = \frac{\eta_{MAX}}{\Delta t \left(1 - \frac{1}{C}\right) + \frac{t^*}{C}} \quad (15)$$

in which  $\eta_0$  represents the corrosion rate assumed for the first branch of the polyline, which, for a given corrosivity category, is only a function of  $\eta_{MAX}$  and  $t^*$ .

For a given corrosion level of interest  $\eta(t)$ , reduced cross-section properties for the element of concern  $A(\eta)$ ,  $W(\eta)$  can be estimated and – through the reasonable assumption of constant internal actions – used to evaluate the stress range amplifications  $k_a$ ,  $k_w$ .

- Damage cumulation is now performed numerically as no closed-form solution can be easily derived. Namely, numerical integration to derive CFF can be done by treating infinitesimal damage  $dD$  as a tiny, yet finite, damage increment  $\Delta D_{i,i+1}$  defined as follows (Equation (16)):

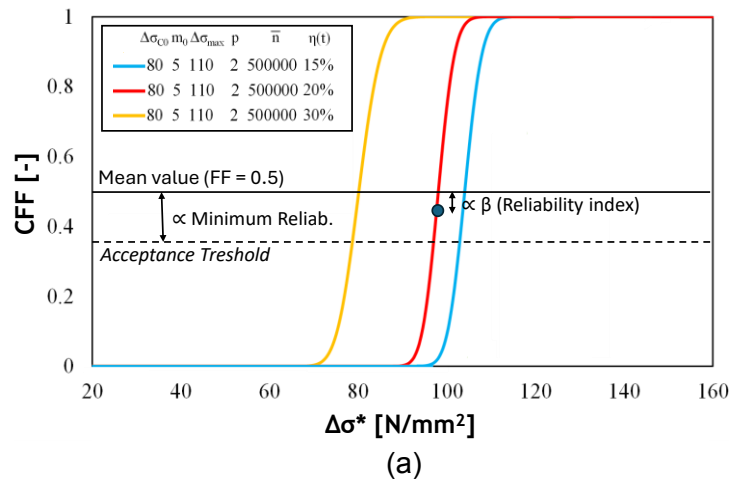
$$\Delta D_{i,i+1} = \frac{k(t_i) [\Delta \sigma^*(\eta_i)]^{b(\eta_i)}}{SSF \cdot A(\eta_i)} \left\{ \left[ 1 - \frac{\log_{10} h_{i+1}}{\log_{10} \bar{h}} \right]^{b(\eta_i)/p} - \left[ 1 - \frac{\log_{10} h_i}{\log_{10} \bar{h}} \right]^{b(\eta_i)/p} \right\} \quad (16)$$

On the basis of the above theoretical framework, corrosion fatigue fragility CFF can be expressed via the following conditioned probability  $P(\text{Failure} \mid \text{Parameters})$  (Equation (17)):

$$CFF = P(D \geq D^* \mid \Delta \sigma^*, p, b, \bar{h}, t, \eta) \quad (17)$$

Based on proposed developments, log-normal distributions for conditioned (corrosion) fatigue failure probability can be obtained for the CI of interest. Therefore, reliability index  $\beta$  can be immediately introduced by recalling its definition as the distance of the demand point  $(\Delta \sigma_c, m, \Delta \sigma^*, p, \bar{h}, \eta)$  from the distribution mean  $\mu$ , i.e., expressed as multiples of the distribution standard deviation  $\sigma$  (that is,  $CFF/FF(\Delta \sigma_c, m, \Delta \sigma^*, p, \bar{h}, \eta) = \mu - \beta \sigma$ ).

Therefore, reliability of existing bridges can be addressed by comparing the derived value of  $\beta$  with a minimum reliability threshold  $\beta_{min}$  to be calibrated based on experimental evidences on real steel bridges (see Figure 5.1-5a). The relevant, optimal maintenance time  $\bar{t}$  can be hence estimated as the instant in which  $\beta(\bar{t}) = \beta_{min}$ .



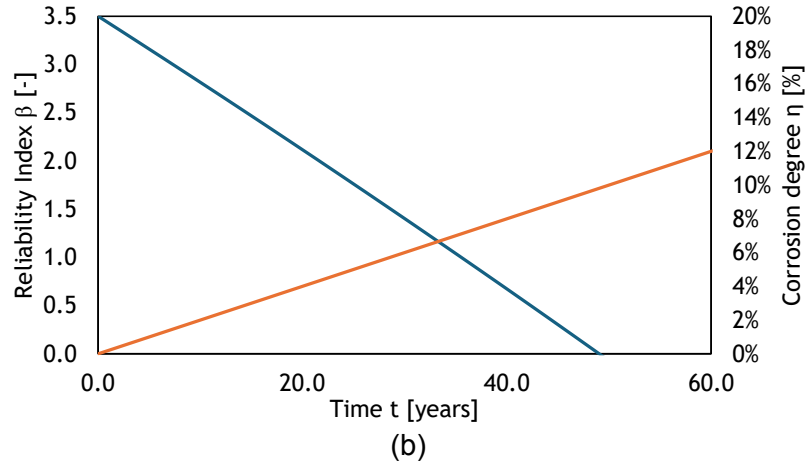


Figure 5.1-5 – (a) Graphical interpretation of reliability index for PM purposes, (b) Typical time-depending trend of corrosion degree and reliability index for a steel bridge.

## 5.2 Software tool for PM of steel bridges

The developed software tool for PM of steel bridges is constituted by an Excel spreadsheet with automated routines written in Visual Basic for Applications (VBA). It is composed of the following worksheets (whose details are thoroughly described in the following):

- Preprocessor for the estimation of the corrosion trend on the basis of measured data;
- Preprocessor for the estimation of cross-section properties' reduction due to corrosion for I or C shaped profiles;
- Input worksheet in which basic information for the case study (traffic distribution, consequence class, detail class for most critical details, moving load analyses results, corrosivity class) are typed in;
- Hidden calculation worksheet in which the above methodologies for PM of steel bridges are applied with reference to four cases: “deterministic” approach in *i*) absence and *ii*) presence of corrosion; probabilistic approach in *iii*) absence and *iv*) presence of corrosion;
- Output worksheet in which relevant graphs (S-N curves, corrosion trends, FF/CFF trends, reliability trends, residual service life trends) are depicted.

### 5.2.1 Preprocessor for the estimation of the corrosion trend

The first spreadsheet features a fillable table in which measured data (if any) in terms of detected corrosion degree ( $\eta$  [%]) against time ( $t$  [years]) can be inputted.

A maximum of 20 in-situ data points can be considered, i.e., provided that the actual corrosion trend is at least non-decreasing (that is,  $t_2 > t_1 \rightarrow \eta_2 \geq \eta_1$  for any  $t_1, t_2$ ). The reference life  $V_{REF}$  for the CI of concern should be also introduced as the upper limit of the corrosion trend. For ease of use, editable fields are highlighted in light blue (see Figure 5.2-1a).

By clicking on the “Optimization” button, corrosion trend parameters (i.e., the corrosivity category  $CX$  and the maximum corrosion degree  $\eta_{max}(t = V_{REF})$ ) are varied according to the least square method, i.e., in order to achieve the minimum quadratic error  $\varepsilon_i^2$  between data points and the regressive bi-linear trend (Equation (18)):

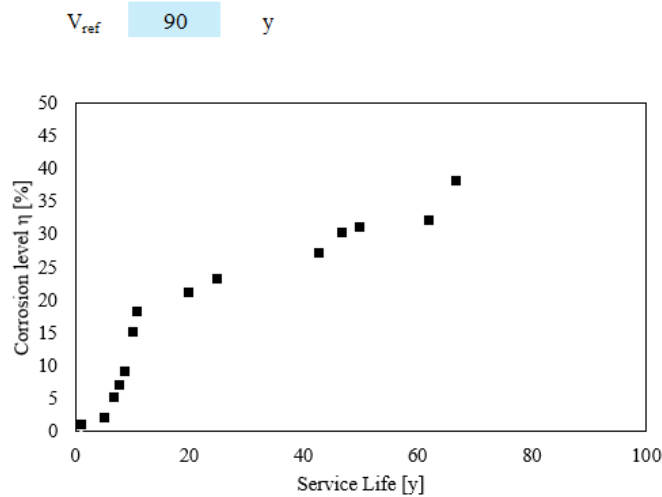
$$\{CX_{opt}; \eta_{max,opt}\} = \arg \min \sum_i \varepsilon_i^2 = \sum_i [\eta_{ISO9224}(t_i) - \eta_{exp}(t_i)]^2 \quad (18)$$

where  $CX_{opt}$ ,  $\eta_{max,opt}$  are the optimized corrosivity category and maximum corrosion degree,  $\eta_{ISO9224}$  is the

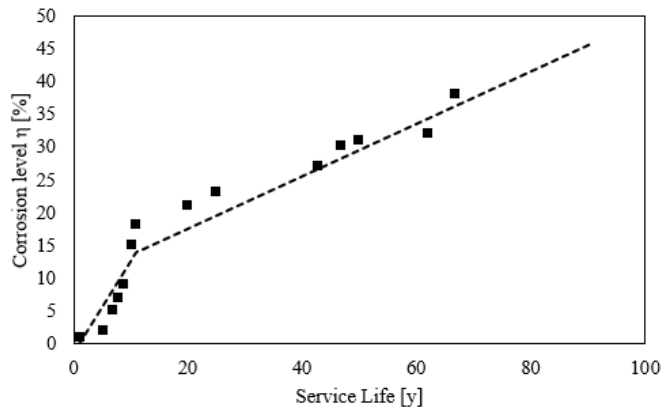
estimated corrosion degree according to the bi-linear approximation (Equations (14)-(15)) and  $\eta_{\text{exp}}$  is the measured corrosion degree. The sum of quadratic squares is extended to the entire set of time inputs  $t_i$  introduced in the fillable Table.

For the sake of completeness, optimized  $\eta_{\text{max}}$  values are outputted for all corrosivity categories, and relevant errors are sorted through a green-to-red scale (see Figure 5.2-1b). Hence, the user may select a different CX for subsequent calculations, e.g., if it was chosen via normative provisions.

| #  | t [y] | $\eta$ [%] |
|----|-------|------------|
| 1  | 1     | 1          |
| 2  | 5     | 2          |
| 3  | 7     | 5          |
| 4  | 8     | 7          |
| 5  | 9     | 9          |
| 6  | 10    | 15         |
| 7  | 11    | 18         |
| 8  | 20    | 21         |
| 9  | 25    | 23         |
| 10 | 43    | 27         |
| 11 | 47    | 30         |
| 12 | 50    | 31         |
| 13 | 62    | 32         |
| 14 | 67    | 38         |
| 15 |       |            |
| 16 |       |            |
| 17 |       |            |
| 18 |       |            |
| 19 |       |            |
| 20 |       |            |



(a)



Optimization

| Class | $\eta_{\text{MAX}}$ [%] | ERR    |
|-------|-------------------------|--------|
| C1    | 42                      | 173.11 |
| C2    | 46                      | 122.52 |
| C3    | 50                      | 135.12 |
| C4    | 54                      | 256.48 |
| C5    | 56                      | 405.07 |

(b)

Figure 5.2-1 – Preprocessing worksheet for the estimation of the corrosion trend: (a) input of measured data, (b) Optimization of parameters for the derivation of the most likely corrosivity class and maximum corrosion degree.

### 5.2.2 Preprocessor for the estimation of cross-section properties' reduction

The second spreadsheet is suited for typical I and C shaped cross sections adopted for girders, stringers, bracings found in steel and composite bridges. Relevant geometric parameters (depth H, width b, flange and web thickness  $t_f$ ,  $t_w$ ) can be inputted in the light blue box.

For ease of use, a large catalogue referred to IPE, HE and UPN profiles is available through a pull-down menu. Nevertheless, geometrical features can still be manually inputted if, for example, non-standard very deep built-up sections are used (see Figure 5.2-2a).

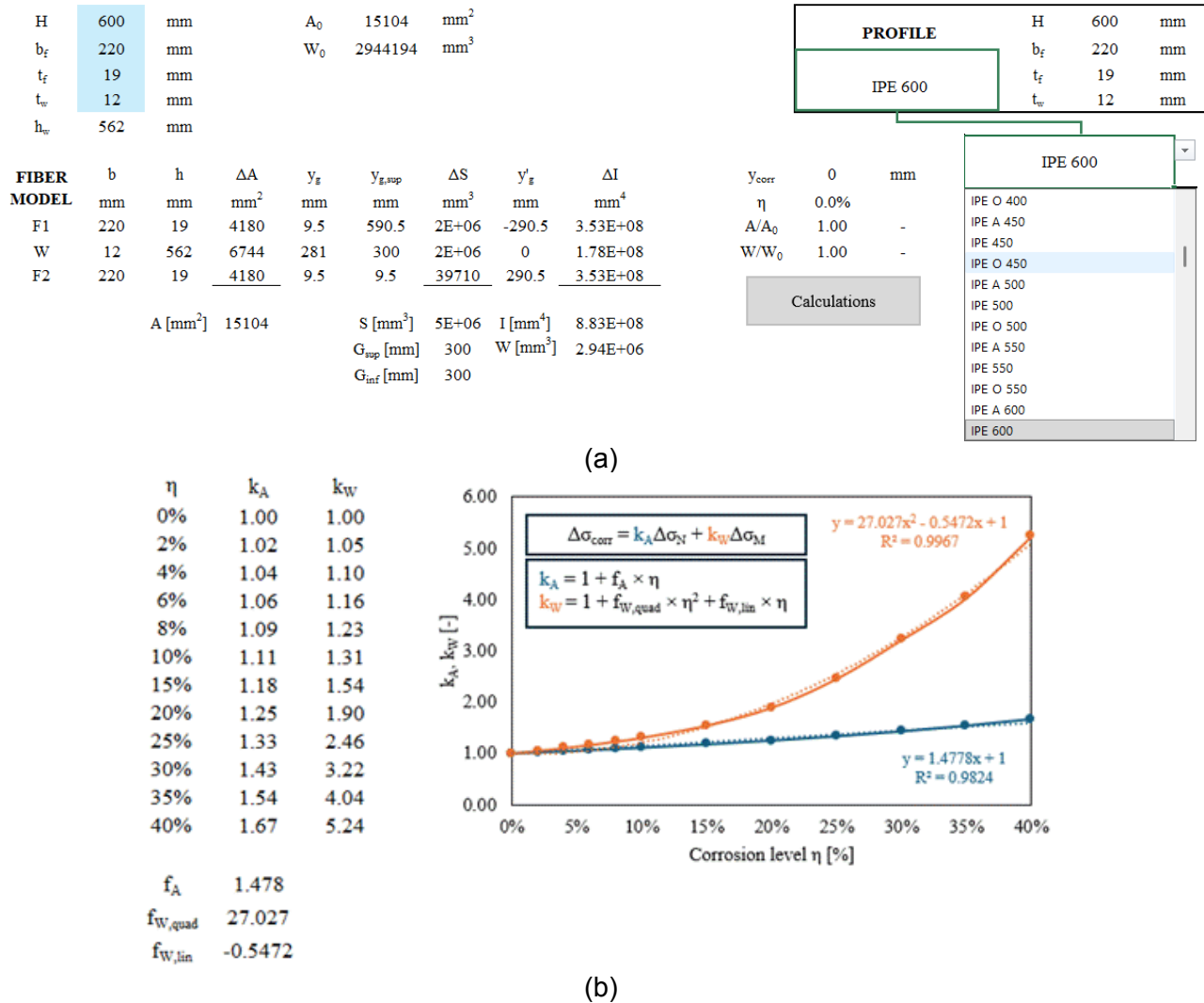


Figure 5.2-2 – Preprocessing worksheet for the estimation of cross-section properties' reduction: (a) input of geometric features – catalogue – fiber model, (b) derivation of trends for stress amplification factors.

By clicking on the “Calculations” button, a fiber model is recursively used for increasing values of the corrosion degree  $\eta$ . Accordingly, *i*) the reduced area  $A(\eta)$ , *ii*) the shifted centroid  $y_g(\eta)$ , *iii*) the reduced second moment of area  $I_Y(\eta)$  and hence *iv*) the reduced elastic modulus  $W_{el,Y}(\eta)$  are estimated according to the following expressions (Equations (19)-(22)):

$$A_i(\eta) = t_i(\eta) \times b_i \quad (19)$$

$$y_g(\eta) = \frac{\sum_i A_i(\eta) \times y_i(\eta)}{\sum_i A_i(\eta)} \quad (20)$$



$$I_Y(\eta) = \sum_i A_i(\eta) \times [y_i(\eta) - y_g(\eta)]^2 \quad (21)$$

$$W_{el,Y}(\eta) = \frac{I_Y(\eta)}{\max[y_g(\eta); H(\eta) - y_g(\eta)]} \quad (22)$$

where  $t_i(\eta)$  is the corrosion-reduced thickness of the  $b_i$ -wide  $i$ -th fiber,  $y_i(\eta)$  is the centroid location of the  $i$ -th fiber and  $H(\eta)$  is the corrosion-reduced depth of the entire cross-section.

According to the assumption of invariable internal actions due to corrosion (that is, mass loss due to corrosion is small enough to avoid internal actions redistributions due to changes in stiffness), the following expression can be used to estimate normal stress amplifications (Equation (23)):

$$\Delta\sigma(\eta) = \frac{\Delta N}{A(\eta)} + \frac{\Delta M}{W_{el,Y}(\eta)} = k_A \Delta\sigma_N + k_W \Delta\sigma_M \quad (23)$$

with  $\Delta N$  and  $\Delta M$  being the axial force range and the bending moment range,  $k_A = A(\eta=0)/A(\eta)$  and  $k_W = W_{el,Y}(\eta=0)/W_{el,Y}(\eta)$ .

In the preprocessor worksheet,  $k_A$  and  $k_W$  trends are estimated by interpolating results referred to some discrete values of  $\eta$  (0, 2, 4, 6, 8, 10, 15, 20, 25, 30, 35, 40%, (see Figure 5.2-2b). Namely, a linear trend is assumed for  $k_A$ , while a quadratic trend is assumed for  $k_W$  based on expected outcomes:

$$k_A \approx 1 + f_a \times \eta \quad (24)$$

$$k_W \approx 1 + f_{W,quad} \times \eta^2 + f_{W,lin} \times \eta \quad (25)$$

where  $f_a$ ,  $f_{W,lin}$ ,  $f_{W,quad}$  are regression coefficients estimated through the least squares method.

### 5.2.3 Input worksheet

The input worksheet is the main interface of the developed tool for PM of steel bridges (see Figure 5.2-3).

The following data are required to run relevant algorithms for damage, probability and reliability calculations:

- **Traffic volume and composition:** reference is always made to FLM4 encoded in both Eurocode 1 – Part 2 and Italian NTC2018, §5. Accordingly, five different equivalent lorries (V1-to-5) are considered and relevant information (number of axles, load per axle, axles spacing) are reported in the attached Table.

The traffic composition should be selected based on the importance of the roadway network in which the CI is placed (i.e., local, medium distance and long distance traffic composition can be assumed). Each composition is associated to fixed percentual frequencies for each lorry (local traffic: 80-5-5-5-5 [%], medium distance: 40-10-30-15-5 [%], long distance: 20-5-50-15-10 [%] for V1, V2, V3, V4 and V5, respectively).

The average yearly heavy traffic is selected based on the so-called traffic category, which depends on the importance of the infrastructure and on the number of slow lanes per direction of travel (high intensity: 2'000'000 trucks/year, medium intensity: 500'000 trucks/year, moderate intensity: 125'000 trucks/year, low intensity: 50'000 trucks/year). By selecting the proper category, the average yearly traffic per each equivalent lorry is hence immediately estimated. The total volume of heavy traffic is estimated as well by accounting for the reference service life;

- **Fatigue capacity:** reference is made to Wohler (or S-N) curves encoded in both Eurocode 3 – Part 1-9 and Italian Guidelines for the application of NTC2018, §C4.2. Accordingly, the detail class  $\Delta\sigma_C$  (i.e., the fatigue strength associated with an expected fatigue life  $N_C = 2'000'000$  cycles) can be selected from a list of 14 alternatives ( $\Delta\sigma_C = 36 \div 160$  MPa) depending on the most critical element to be checked against (corrosion) fatigue. Relevant values of  $\Delta\sigma_D (= 0.737 \Delta\sigma_C$ , constant amplitude fatigue limit CAFL,  $N_D = 5'000'000$  cycles) and  $\Delta\sigma_L (= 0.405 \Delta\sigma_C$ , cutoff limit COL,  $N_F = 100'000'000$  cycles) are also immediately estimated accounting for S-N slopes



prescribed by EN1993-1-9 (i.e.,  $m = 3$  for  $N \leq N_D$ ,  $m = 5$  for  $N_D \leq N \leq N_F$ ,  $m \rightarrow \infty$  for  $N > N_F$ ). The relevant partial safety factor for resistance ( $\gamma_{MF} = 1.00 \div 1.15$ ) is also accounted for based on expected fatigue collapse consequences (low or high);

- **Fatigue demand:** the load spectrum deriving from moving load analysis should be inputted in terms of stress ranges deriving from oscillating axial forces ( $\Delta\sigma_N$ ) and bending moments ( $\Delta\sigma_M$ ) for each vehicle V1-to-5;
- **Exposure to corrosion:** the corrosivity category CX and the maximum expected corrosion degree  $\eta_{\max}(t = V_{REF})$  can be inputted based on results from preprocessing.

By clicking on the lower button, relevant VBA macros in the hidden calculation worksheet are launched based on inputted data.

| FATIGUE LOAD MODEL N°4 (Tab. 5.1.VIII NTC 2018) |                                 |                              |                                          |                           |                   |                 | TRAFFIC CATEGORY (Tab 5.1.X NTC 2018)                                                                       |                                                                                 |
|-------------------------------------------------|---------------------------------|------------------------------|------------------------------------------|---------------------------|-------------------|-----------------|-------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------|
| Sagoma del veicolo                              | Tipo di pneumatico (Tab.5.1-IX) | Intensità [m]                | Valori equivalenti dei carichi asse [kN] | COMPOSIZIONE DEL TRAFFICO |                   |                 | Categorie di traffico                                                                                       | Flusso annuo di veicoli di peso superiore a 100 kN sulla corsia di marcia lenta |
|                                                 |                                 |                              |                                          | Lunga percorrenza         | Media percorrenza | Traffico locale |                                                                                                             |                                                                                 |
| V1                                              | A<br>B                          | 4,50                         | 70<br>130                                | 20,0                      | 40,0              | 80,0            | 1 - Strade ed autostrade con 2 o più corsie per senso di marcia, caratterizzate da intenso traffico pesante | 2,0x10 <sup>6</sup>                                                             |
| V2                                              | A<br>B<br>B                     | 4,20<br>1,30                 | 70<br>120<br>120                         | 5,0                       | 10,0              | 5,0             | 2 - Strade ed autostrade caratterizzate da traffico pesante di media intensità                              | 0,5x10 <sup>6</sup>                                                             |
| V3                                              | A<br>B<br>C<br>C<br>C           | 3,20<br>5,20<br>1,30<br>1,30 | 70<br>150<br>90<br>90<br>90              | 50,0                      | 30,0              | 5,0             | 3 - Strade principali caratterizzate da traffico pesante di modesta intensità                               | 0,125x10 <sup>6</sup>                                                           |
| V4                                              | A<br>B<br>B<br>B                | 3,40<br>6,00<br>1,80         | 70<br>140<br>90<br>90                    | 15,0                      | 15,0              | 5,0             | 4 - Strade locali caratterizzate da traffico pesante di intensità molto ridotta                             | 0,05x10 <sup>6</sup>                                                            |
| V5                                              | A<br>B<br>C<br>C<br>C           | 4,80<br>3,60<br>4,40<br>1,30 | 70<br>130<br>90<br>80<br>80              | 10,0                      | 5,0               | 5,0             |                                                                                                             |                                                                                 |

| COMPOSITION    |       |   |  |
|----------------|-------|---|--|
|                | Media |   |  |
| n <sub>1</sub> | 40    | % |  |
| n <sub>2</sub> | 10    | % |  |
| n <sub>3</sub> | 30    | % |  |
| n <sub>4</sub> | 15    | % |  |
| n <sub>5</sub> | 5     | % |  |

| CATEGORY           |           |       |  |
|--------------------|-----------|-------|--|
|                    | Intensità | Media |  |
| N <sub>TOT,y</sub> | 5.00E+05  | v/y   |  |
| n <sub>1</sub>     | 2.00E+05  | v/y   |  |
| n <sub>2</sub>     | 8.00E+04  | v/y   |  |
| n <sub>3</sub>     | 3.20E+04  | v/y   |  |
| n <sub>4</sub>     | 1.28E+04  | v/y   |  |
| n <sub>5</sub>     | 5.12E+03  | v/y   |  |

| TRAFFIC DATA     |          |   |  |
|------------------|----------|---|--|
| V <sub>R</sub>   | 100      | y |  |
| N <sub>TOT</sub> | 5.00E+07 | v |  |

| REFERENCE LIFE |     |   |  |
|----------------|-----|---|--|
| V <sub>R</sub> | 100 | y |  |

| TOTAL TRAFFIC    |          |   |  |
|------------------|----------|---|--|
| N <sub>TOT</sub> | 5.00E+07 | v |  |

1 Detail category  $\Delta\sigma_C$   
2 Constant amplitude fatigue limit  $\Delta\sigma_D$   
3 Cut-off limit  $\Delta\sigma_L$

| FATIGUE ANALYSIS   |                  |           |                   |
|--------------------|------------------|-----------|-------------------|
| DETAIL CLASS       | $\Delta\sigma_C$ | 71        | N/mm <sup>2</sup> |
|                    | N <sub>C</sub>   | 2000000   | cicli             |
|                    | N <sub>D</sub>   | 5000000   | cicli             |
|                    | N <sub>L</sub>   | 100000000 | cicli             |
| FATIGUE PARAMETERS | m <sub>1</sub>   | 3         | -                 |
|                    | m <sub>2</sub>   | 5         | -                 |
|                    | $\Delta\sigma_D$ | 52.3      | N/mm <sup>2</sup> |
|                    | $\Delta\sigma_L$ | 28.7      | N/mm <sup>2</sup> |

| PARTIAL SAFETY F. |                       |       |   |
|-------------------|-----------------------|-------|---|
|                   | Collapse Consequences | Basse |   |
|                   | $\gamma_{MF}$         | 1.00  | - |

| LOAD SPECTRUM |                     |      |                   |
|---------------|---------------------|------|-------------------|
| V1            | $\Delta\sigma_{1N}$ | 0.4  | N/mm <sup>2</sup> |
|               | $\Delta\sigma_{1M}$ | 9.3  | N/mm <sup>2</sup> |
| V2            | $\Delta\sigma_{2N}$ | 0.6  | N/mm <sup>2</sup> |
|               | $\Delta\sigma_{2M}$ | 14.4 | N/mm <sup>2</sup> |
| V3            | $\Delta\sigma_{3N}$ | 1.0  | N/mm <sup>2</sup> |
|               | $\Delta\sigma_{3M}$ | 21.3 | N/mm <sup>2</sup> |
| V4            | $\Delta\sigma_{4N}$ | 0.8  | N/mm <sup>2</sup> |
|               | $\Delta\sigma_{4M}$ | 16.7 | N/mm <sup>2</sup> |
| V5            | $\Delta\sigma_{5N}$ | 0.9  | N/mm <sup>2</sup> |
|               | $\Delta\sigma_{5M}$ | 19.0 | N/mm <sup>2</sup> |

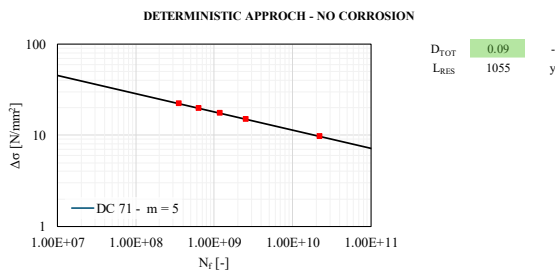
| CORROSION         |      |
|-------------------|------|
| Corrosivity Class | C2   |
| $\eta_{\max}$     | 46.0 |
|                   | %    |

Figure 5.2-3 – Main input worksheet.

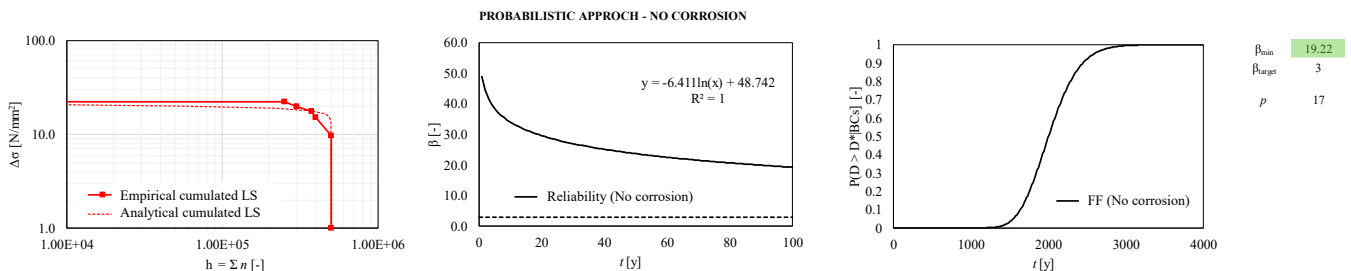
## 5.2.4 Output worksheet

The output worksheet yields all relevant information for PM of steel bridges in the form of both graphs and indicators (see Figure 5.2-4). Namely:

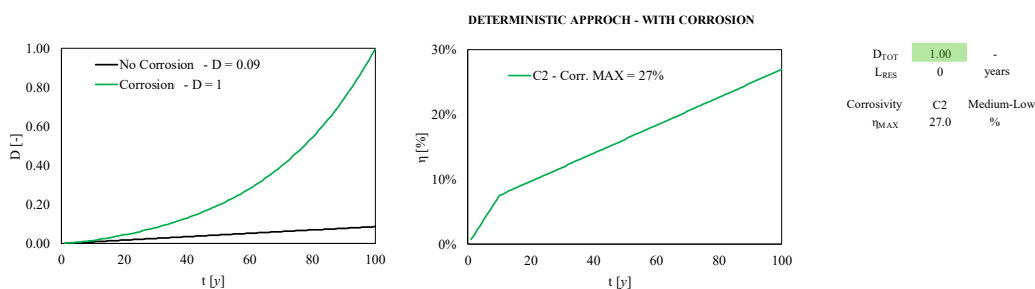
- For the deterministic (or, more properly, semi-probabilistic) approach with no corrosion (i.e., the “standard” normative approach for fatigue checks), the S-N curve of interest with demand points is provided, i.e., in conjunction with the total damage  $D_{TOT}$  and the residual service life  $L_{RES}$  (see Figure 5.2-4a);
- For the probabilistic approach with no corrosion, the approximated load spectrum and the reliability and FF trends against time are provided, i.e., in conjunction with the minimum reliability index  $\beta_{min}(t = V_{REF})$ , the shape parameter  $p$  and the residual service life  $L_{RES}$  (see Figure 5.2-4b);
- For the deterministic approach with corrosion, the damage and corrosion trends against time are provided, i.e., in conjunction the total damage  $D_{TOT}$  and the residual service life  $L_{RES}$  (see Figure 5.2-4c). If the relevant button is clicked, the critical corrosion degree  $\eta^*$  leading to collapse ( $D = 1$ ) for  $t = V_{REF}$  can be calculated;
- For the probabilistic approach with corrosion (i.e., the most general approach), the reliability and CFF trends against time are provided, i.e., in conjunction with the minimum reliability index  $\beta_{min}(t = V_{REF})$ , the residual service life  $L_{RES}$  and the equivalent, corrosion-reduced fatigue strength (see Figure 5.2-4d). If the relevant button is clicked, the critical corrosion degree  $\eta^*$  leading to collapse ( $\beta = \beta_{min}$ ) for  $t = V_{REF}$  can be calculated.



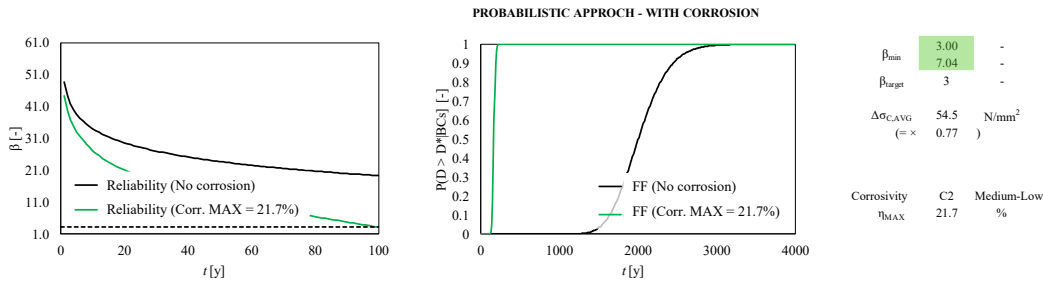
(a) deterministic approach, no corrosion (normative approach for fatigue)



(b) probabilistic approach, no corrosion



(c) deterministic approach, with corrosion



(d) probabilistic approach, with corrosion (most general approach)

Figure 5.2-4 – Output worksheet.

## 5.3 Integration with IT Platform at System Level

The developed software tool for PM of steel bridges features an export function which was implemented through a dedicated VBA macro.

Namely, main results collected in the Output worksheet can be exported in a comma-separated values (.csv) file in which the following main information is given in a Table (see Table 5.3-1):

Table 5.3-1 – Outputs provided in the exported .csv file.

| DATA TYPE                  | FIELD                 | VALUE                | DESCRIPTION                                            |
|----------------------------|-----------------------|----------------------|--------------------------------------------------------|
| General Information        | ID                    | 1, 2, ..., i, ..., n | Label of the viaduct (Integer)                         |
|                            | Name                  | XXXXXXXXXX           | Name of the viaduct (String)                           |
|                            | Service Life          | XXX years            | Reference service life as per EN1990 (Integer)         |
|                            | Year of evaluation    | XXX years            | Evaluation time for reported outputs (Integer)         |
| Traffic Data               | Traffic category      | Local, Medium, Long  | cfr. Tab. 5.1.VIII NTC2018 (String)                    |
|                            | AYT                   | 50'000 ÷ 200'000     | cfr. Tab. 5.1.X NTC2018 (Integer)                      |
| Fatigue Data               | Detail Class          | 36 ÷ 160 MPa         | Detail class for fatigue $\Delta\sigma_C$ (Integer)    |
|                            | Consequence Class     | Low, High            | CC to select $\gamma_{MF} = 1.00/1.15$ (String)        |
| Load Spectrum              | $\Delta\sigma_{N,V1}$ | X.X MPa              | $\Delta\sigma$ for Axial Force, V1 (Float)             |
|                            | $\Delta\sigma_{M,V1}$ | X.X MPa              | $\Delta\sigma$ for Bending Moment, V1 (Float)          |
|                            | $\Delta\sigma_{N,V2}$ | X.X MPa              | $\Delta\sigma$ for Axial Force, V2 (Float)             |
|                            | $\Delta\sigma_{M,V2}$ | X.X MPa              | $\Delta\sigma$ for Bending Moment, V2 (Float)          |
|                            | $\Delta\sigma_{N,V3}$ | X.X MPa              | $\Delta\sigma$ for Axial Force, V3 (Float)             |
|                            | $\Delta\sigma_{M,V3}$ | X.X MPa              | $\Delta\sigma$ for Bending Moment, V3 (Float)          |
|                            | $\Delta\sigma_{N,V4}$ | X.X MPa              | $\Delta\sigma$ for Axial Force, V4 (Float)             |
|                            | $\Delta\sigma_{M,V4}$ | X.X MPa              | $\Delta\sigma$ for Bending Moment, V4 (Float)          |
|                            | $\Delta\sigma_{N,V5}$ | X.X MPa              | $\Delta\sigma$ for Axial Force, V5 (Float)             |
|                            | $\Delta\sigma_{M,V5}$ | X.X MPa              | $\Delta\sigma$ for Bending Moment, V5 (Float)          |
| Corrosion Modelling        | $f_A$                 | X.XXX                | Linear regr. coeff. for Area Loss (Float)              |
|                            | $f_{W,lin}$           | X.XXX                | Linear regr. coeff. for Modulus Loss (Float)           |
|                            | $f_{W,quad}$          | X.XXX                | Quadratic regr. coeff. for Modulus Loss (Float)        |
|                            | CX                    | C1, C2, C3, C4, C5   | Corrosivity Category as per ISO9224 (String)           |
|                            | $\eta_{MAX}$          | X.X %                | Maximum corrosion degree (Float, [%])                  |
| Deterministic No Corrosion | $D_{TOT,NC}$          | X.XX                 | Total damage w/out corrosion (Float)                   |
|                            | $L_{RES,DET,NC}$      | XXXX years           | Residual life w/out corrosion - determ. (Integer)      |
| Probabilistic No Corrosion | $\beta_{MIN,NC}$      | X.XX                 | Residual reliability w/out corrosion - determ. (Float) |
|                            | $L_{RES,PROB,NC}$     | XXXX years           | Residual life w/out corrosion - prob. (Integer)        |
| Deterministic Corrosion    | $D_{TOT,C}$           | X.XX                 | Total damage with corrosion (Float)                    |
|                            | $L_{RES,DET,C}$       | XXXX years           | Residual service life with corrosion (Integer)         |
| Probabilistic Corrosion    | $\beta_{MIN,C}$       | X.XX                 | Residual reliability with corrosion (Float)            |
|                            | $\eta_{CRIT}$         | X.X %                | Critical corrosion degree (Float, [%])                 |

The above data can be easily imported in an IT, gis-based platform to monitor the structural health of multiple CIs. This also enables the infrastructures' manager to define a rank of priorities for retrofit interventions, e.g., based on the provided information on vulnerability which should be crossed with relevant, global scale, data in terms of multi-hazard (including other sources of damage such as earthquakes, landslides, floods...) and exposition.

## 5.4 Applicative example: POC2 Cadore Bridge

### 5.4.1 Generalities

The Cadore bridge is a 272 m long, full-steel orthotropic steel deck (OSD) bridge. This bridge belongs to a series of viaducts erected to improve the viability on the Alemagna State Road 51 (i.e., one of the CIs selected as POC2 within RETURN project) between mountainous areas of Treviso, Belluno and Bolzano provinces. Cadore bridge crosses the Piave river in the locality of Perarolo di Cadore (BL – see Figure 5.4-1).



Figure 5.4-1 – Geolocalization and lateral view of the Cadore Bridge in Perarolo di Cadore (BL).

Being designed by Matildi Engineering firm and built by the Cimolai Costruzioni company in about three years – Cadore bridge was opened to traffic in 1985. The relevant, global structural scheme involves an arch-frame arrangement, i.e., with two pairs of box-shaped steel piers inclined of  $\approx 40^\circ$ , ( $H = 55$  m, variable section).

The deck is divided into a central span ( $L = 128$  m) and two lateral spans ( $L = 72$  m). It features a double box section with variable dimensions along the bridge length ( $H = 6$  m). The upper box constituting the road platform is composed by a 13.3 m wide open-stiffener orthotropic slab, the upper plate of which is in direct contact with the asphalt layer without the presence of any concrete slab. All the described plate elements and profiles described above are made of self-passivating steel.

### 5.4.2 Constructional details

Due to well-known complex geological conditions, Cadore bridge foundations had to be conceived as light as possible. This, in conjunction with the considerable span to be crossed (272 m), lead to the definition of the mentioned arch-frame scheme in place of a bulkier arch scheme.

The longitudinal bridge profile exhibits a slope of  $\approx 5\%$  ( $\Delta H \approx 15$  m among the two ends of the bridge). The structure is restrained on four sides, i.e., at the base of each couple of piers and at the deck ends. In the first case, fixed hinge restraints were adopted, and appropriate structural details were developed (see Figure 5.4-2a). As for deck restraints, a fixed hinge was adopted on Tai side, while the other side was equipped with a roller, i.e., leaving the deck free "to breathe" under thermic excursions.

Transferring of horizontal actions from deck to piers to abutments was aided through special girders (steel-concrete composite “tie-girders” equipped with anti-seismic devices), which were also crucial in the assembly phase of both the pier legs and relative semi-girders.

As for cross-sections of the arch-frame system, the deck girder features a double box section ( $W = 6.60$  m,  $H = 6$  m in the central span,  $H = 3 \div 6$  m in side spans – see Figure 5.4-2b). The 13.3 m wide is made with an open-stiffener orthotropic plate composed by transverse sheets spaced at a distance of 3 m and 0.3 m spaced flat ribs placed longitudinally. In the same fashion, the webs and the bottom flange of the lower box (core) are also stiffened by transverse sheets ( $p = 3$  m) and longitudinal flat ribs (webs)/half-IPE segments (flange). Transverse stiffening system is completed by X-braces made of coupled L profiles, ( $p = 6$  m).



Figure 5.4-2 – Structural details of Cadore bridge: a) base hinge, b) central span OSD.

With regards to piers, each one is made up of a pair of legs inclined along both main axes (longitudinal slope  $\approx 40^\circ$ , transverse opening at the base  $\approx 19.2$  m). Legs are connected by a box-shaped transverse rod placed at about  $\frac{1}{3} H_{\text{pier}}$  from the hinges ( $4 \times 3$  m). Each leg features a rectangular hollow section ( $H = 3 \div 6$  m increasing towards the deck,  $W = 2$  m) being stiffened through flat ribs ( $p = 0.3$  m) on all internal sides. 6 m-pitched Z-braces are present also in this case.

Both girder and piers were conceived as an assemblage of multiple, 6 m long segments due to shipment reasons. Structural continuity was hence restored on-site through double covered joints equipped with high-strength bolts. In a similar fashion, girder-to-legs connections were made through welded special segments at the piers' upper ends which were in turn bolted to the adjacent deck segments.

#### 5.4.3 Geometric reconstruction of POC2 Cadore bridge

On the basis of data provided by the bridge manager and recollected from literature, a thorough geometric reconstruction of Cadore bridge was carried out. Namely, global dimensions, plates' thicknesses and configuration of stiffeners were derived for each deck and pier segment.

As a consequence of structural optimization, 10 different cross-section configurations were adopted for the girder (see Figure 5.4-3, blue markers), while 8 configurations were adopted for each pier (see Figure 5.4-3, cyan markers).

With reference to the specific purpose of the following developments (corrosion fatigue analysis and study of the effectiveness of low-impact retrofit strategies), reconstructed details were rationally simplified, i.e., in order to derive computationally addressable cross-section configurations for which

mechanical properties could be easily estimated and manipulated to account for material degradation, if relevant.

An example of application of such logic is depicted in Figure 5.4-4 with reference to the middle girder segment (Cross-section A-A' – Figure 5.4-4a) and the middle pier leg segment (Cross-section D-47 – Figure 5.4-4b).

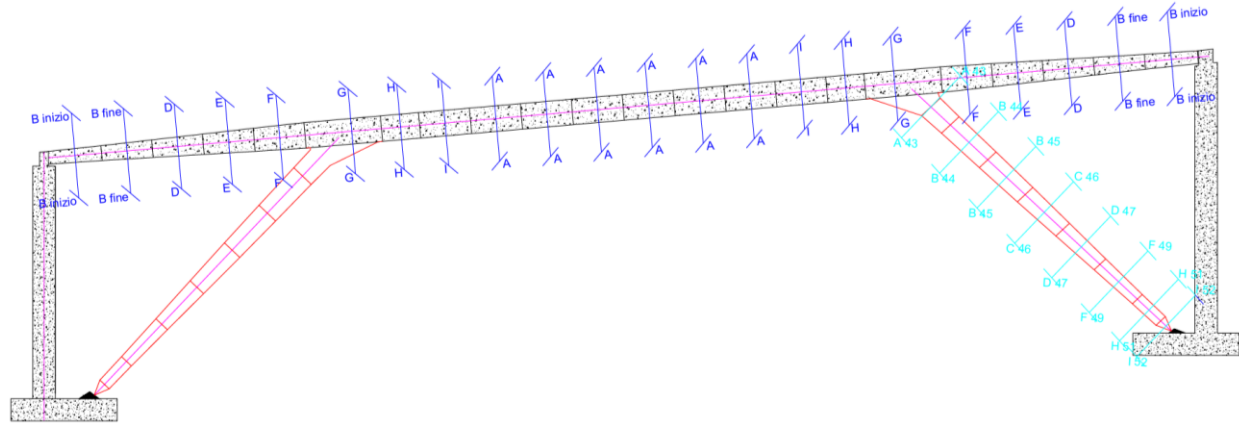


Figure 5.4-3: Structural lateral view of the Cadore bridge with different cross-section configurations being highlighted in blue (girder) and cyan (piers).

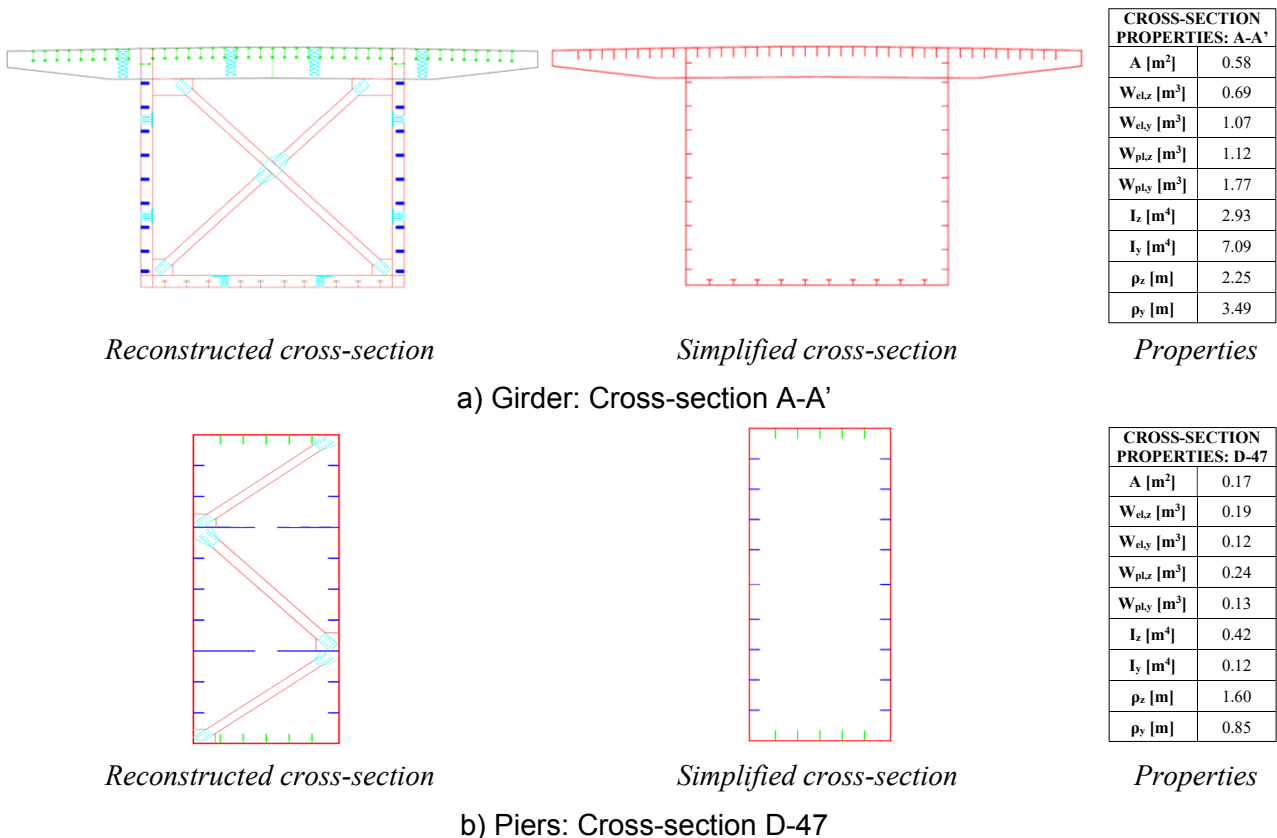


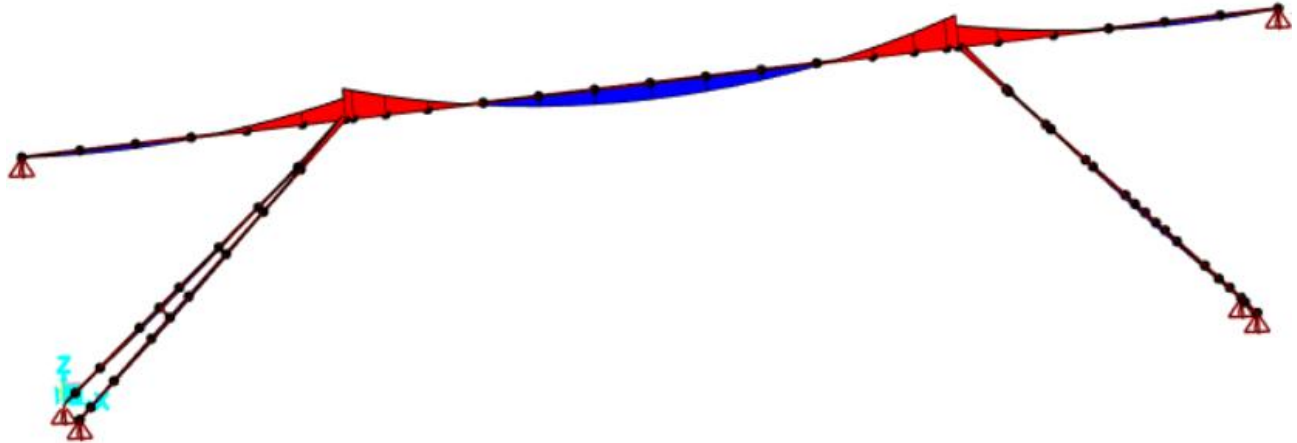
Figure 5.4-4: Examples of cross-section modelling for the Cadore bridge.

#### 5.4.4 Structural modelling of POC2 Cadore bridge

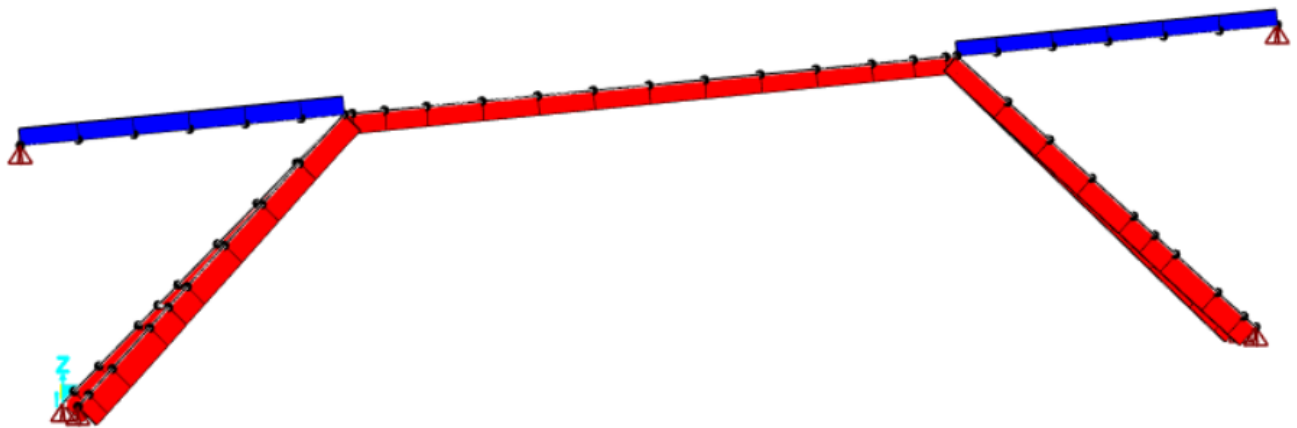
Structural modelling of the Cadore bridge was carried out in SAP2000 environment. Each girder and pier segment was modelled through a 1D frame element being aligned with the relevant cross-section centroid. In compliance with the intended response of structural details, no releases were

placed in the connection zone among piers' legs and the girder (full-rigid joints). In the same fashion, base joints and deck ends' detailing were modelled through equivalent hinge or roller restraints.

Validity of developed structural modelling is proved in Figure 5.4-5, in which diagrams of bending moment  $M(s)$  and axial force  $N(s)$  are depicted under ULS permanent loads combination (self weight  $g_{1k}$  + asphalt and barriers  $g_{2k}$ ). As expected, the full-rigidity of pier-to-girder joints results in a slight jump discontinuity of  $M(s)$  along the deck. Moreover, the tie-action exploited by side spans is clearly visible, while the central arch scheme is rightly fully compressed.



a) Bending moment  $M(s)$



b) Axial Force  $N(s)$

Figure 5.4-5: Internal action diagrams for ULS permanent loads combination:  
a) bending moment  $M$ , b) axial force  $N$ .

#### 5.4.5 Influence lines

In order to address (corrosion) fatigue calculations, influence lines in performance-governing sections  $S$  were derived. For this purpose, vertical unitary forces  $F_Y = 1$  were considered as moving entities on a path coinciding with the roadway profile.

For the estimation of normal stress ranges  $\Delta\sigma$  from relevant oscillograms  $\sigma(t)$ , both bending moment  $M_{S^{FY}}$  and axial force  $N_{S^{FY}}$  influence lines were derived.

As an example, Figure 5.4-6 depicts:

- The bending moment influence line  $M_{S^{FY}}$  for the OSD leg-to-girder nodal section (F-F' – Figure 5.4-6a);

- The axial force influence line  $N_S^{F_Y}$  for the pier base section (H-54 – Figure 5.4-6b);

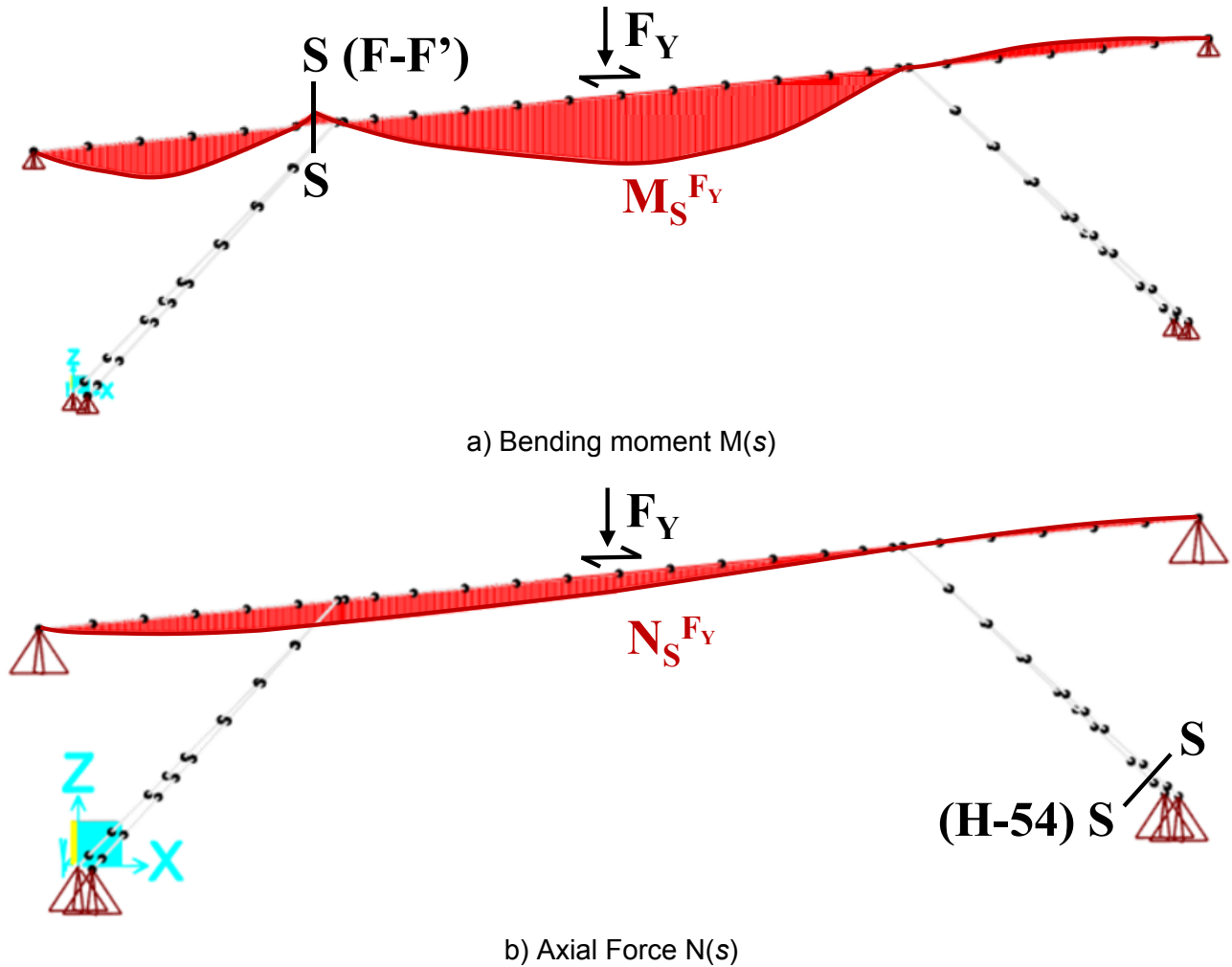


Figure 5.4-6: Influence lines for vertical moving loads running on the bridge deck:  
a) bending moment  $M_S^{F_Y}$  in F-F', b) axial force  $N_S^{F_Y}$  in H-54.

#### 5.4.6 Fatigue loads

In light of known features about the SS51 road transportation network, a “medium distance” composition of traffic was assumed for calculations (see Figure 5.4-7).

As for the total number of heavy trucks per year (average yearly traffic AYT),  $n = 500000$  vehicles/year was assumed in compliance with NTC2018, Tab. 5.1.X, i.e., considering a moderate flow of heavy trucks due to the presence of a single slow lane per travel direction.

#### 5.4.7 Stress oscillograms in absence of degradation phenomena

By combining information referred to the bridge structural behavior (influence lines) and to the composition of traffic loads (FLM4), normal stress oscillograms for each cross-section of interest were first derived under the assumption of null material degradation.

As an example,  $\sigma(t)$  diagrams referred to deck section F-F' and for each equivalent lorry are depicted in Figure 5.4-8.

Tab. 5.1.X – Flusso annuo di veicoli pesanti sulla corsia di marcia lenta

| Categorie di traffico                                                                                       | Flusso annuo di veicoli di peso superiore a 100 kN sulla corsia di marcia lenta |
|-------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------|
| 1 - Strade ed autostrade con 2 o più corsie per senso di marcia, caratterizzate da intenso traffico pesante | 2,0x10 <sup>6</sup>                                                             |
| 2 - Strade ed autostrade caratterizzate da traffico pesante di media intensità                              | 0,5x10 <sup>6</sup>                                                             |
| 3 - Strade principali caratterizzate da traffico pesante di modesta intensità                               | 0,125x10 <sup>6</sup>                                                           |
| 4 - Strade locali caratterizzate da traffico pesante di intensità molto ridotta                             | 0,05x10 <sup>6</sup>                                                            |

Tab. 5.1.VIII - Modello di carico di fatica 4 – veicoli equivalenti

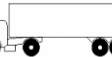
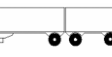
| Sagoma del veicolo                                                                     | Tipo di pneumatico (Tab.5.1.IX) | Interassi [m] | Valori equivalenti dei carichi asse [kN] | COMPOSIZIONE DEL TRAFFICO |                   |                 |
|----------------------------------------------------------------------------------------|---------------------------------|---------------|------------------------------------------|---------------------------|-------------------|-----------------|
|                                                                                        |                                 |               |                                          | Lunga percorrenza         | Media percorrenza | Traffico locale |
| V1    | A                               | 4,50          | 70                                       | 20,0                      | 40,0              | 80,0            |
|                                                                                        | B                               |               | 130                                      |                           |                   |                 |
| V2    | A                               | 4,20          | 70                                       | 5,0                       | 10,0              | 5,0             |
|                                                                                        | B                               |               | 120                                      |                           |                   |                 |
|                                                                                        | B                               |               | 120                                      |                           |                   |                 |
| V3    | A                               | 3,20          | 70                                       | 50,0                      | 30,0              | 5,0             |
|                                                                                        | B                               |               | 150                                      |                           |                   |                 |
|                                                                                        | C                               |               | 90                                       |                           |                   |                 |
|                                                                                        | C                               |               | 90                                       |                           |                   |                 |
| V4    | A                               | 3,40          | 70                                       | 15,0                      | 15,0              | 5,0             |
|                                                                                        | B                               |               | 140                                      |                           |                   |                 |
|                                                                                        | B                               |               | 90                                       |                           |                   |                 |
|                                                                                        | B                               |               | 90                                       |                           |                   |                 |
| V5  | A                               | 4,80          | 70                                       | 10,0                      | 5,0               | 5,0             |
|                                                                                        | B                               |               | 130                                      |                           |                   |                 |
|                                                                                        | C                               |               | 90                                       |                           |                   |                 |
|                                                                                        | C                               |               | 80                                       |                           |                   |                 |

Figure 5.4-7: Assumed fatigue loads and relevant distributions according to Italian NTC2018.

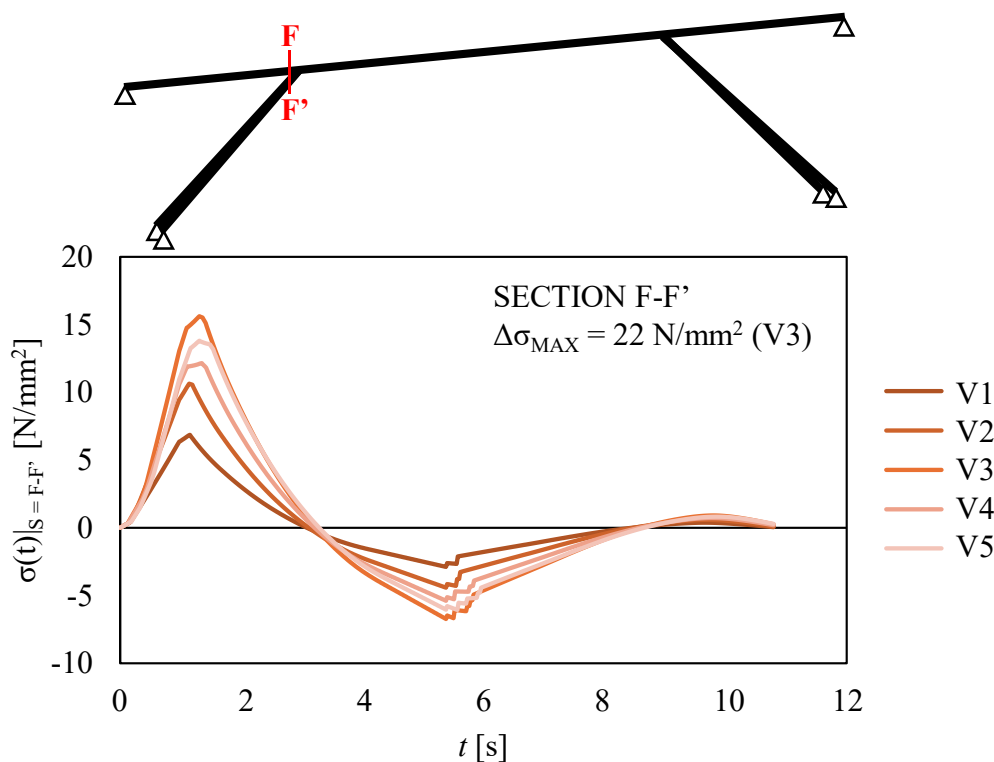


Figure 5.4-8: NTC2018-compliant stress oscillograms for deck section F-F'.

#### 5.4.8 Modelling of corrosion-affected fatigue demand

As stated previously, all plates and members of the Cadore bridge viaduct are made of weathering steel. This choice, in principle, results in a superior corrosion performance of the structure due to the well-known process of formation of a protective (passivated) layer on exposed surfaces.

Nevertheless, as reported in scientific literature, in presence of multiple factors such as *i*) persistent moisture, *ii*) reduced ventilation and *iii*) insufficient maintenance, even weathering steel components can deteriorate due to material loss.

For the relevant case of Cadore bridge, these condition can potentially occur due to the peculiar configurations of cross-sections, i.e., closed boxes in which internal ventilation is limited and meteoric water can leak in from continuity joints and stagnate on the bottom. As a result, localized corrosive phenomena may – for example – occur on deck lower flanges. To this end, it is noteworthy reporting that even very large levels of local corrosion  $\eta_{\text{local},i} = \Delta A_{\text{plate,corr},i} / A_{\text{plate},i}$  on a given *i*-th plate only result in very moderate corresponding global corrosion levels  $\eta_{\text{global}} = \Delta A_{\text{corr}} / A_{\text{TOT}}$ .

The above condition justifies the adopted approach in which global stiffness reduction due to corrosive phenomena was neglected for any level of  $\eta_{\text{global}}$ , while punctual cross-section properties of interest  $A_{S,\text{corr}} / W_{\text{el},Y,S,\text{corr}}$  were assumed as a function of corrosion level. This enabled performing moving load analyses only on the pristine configuration of Cadore bridge, and corrosion was hence accounted – when relevant – by scaling oscillograms through the already introduced  $k_A$ ,  $k_W = f(\eta(t))$  factors.

An example of application of the described technique is illustrated in Figure 5.4-9 with respect to deck section F-F'. Accordingly, medium-large levels of localized corrosion (up to 10%) on the lower flange (initial thickness  $t = 10$  mm) only result in very small levels of global corrosion ( $\approx 2\%$ ). The corresponding increase of axial force-induced normal stresses  $\sigma_N(t, \eta(t))$  is hence very limited ( $f_A = 0.2336$ ).

Contrariwise, the increase of bending-induced normal stresses  $\sigma_M(t, \eta(t))$  is more significant, i.e., due to the lower flange being the most centrifugated element of the cross-section with respect to *y*-*y* axis ( $f_{W,\text{lin}} = 1.15416$ ,  $f_{W,\text{quad}} = 0$ ).

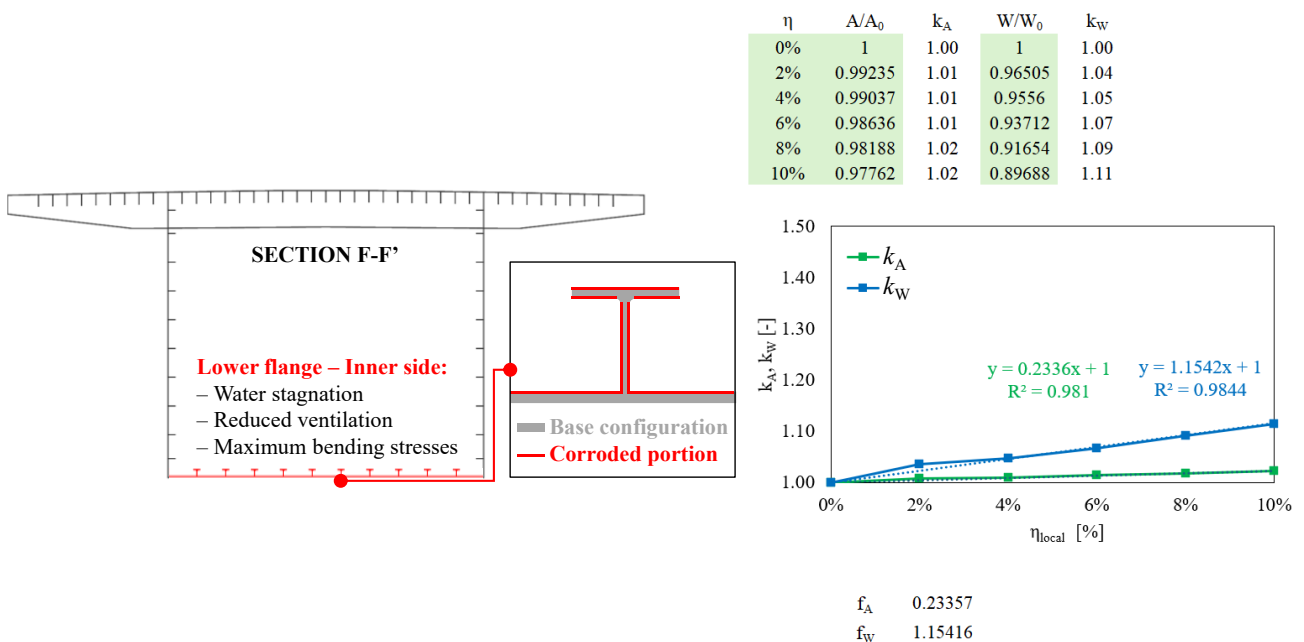
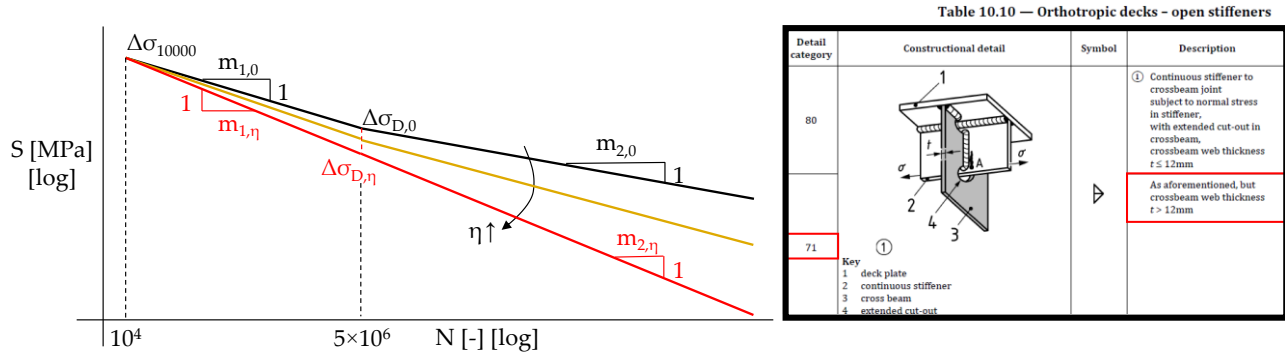


Figure 5.4-9: Example of corrosion modelling for deck section F-F'.

#### 5.4.9 Modelling of corrosion-affected fatigue capacity

Based on normative indications for the pristine detail of interest (open OSD stiffener, crossbeam thicker than 12 mm → FAT 71), corrosion-affected S-N- $\eta$  domains were estimated on the basis of the mentioned Adasooriya et al. model (see Figure 5.4-10).



| Conditions    | Detail Class $\Delta\sigma_C$ | CAFL $\Delta\sigma_D$ | First slope $m_1$ | Second slope $m_2$ |
|---------------|-------------------------------|-----------------------|-------------------|--------------------|
| Pristine      | 71.0 MPa                      | 52.2 MPa              | 3.00              | 5.00               |
| $\eta = 5\%$  | 67.0 MPa                      | 48.9 MPa              | 2.91              | 4.91               |
| $\eta = 10\%$ | 63.2 MPa                      | 45.6 MPa              | 2.81              | 4.81               |

Figure 5.4-10: Corrosion fatigue domains adopted for the assessment of Cadore bridge.

#### 5.4.10 Relevant outputs and discussion

Figure 5.4-11 graphically summarized fatigue checks as per NTC2018 for the most critical welded detail from Cadore bridge (open stiffeners' welds, section F-F).

It can be easily noticed that the total fatigue damage is very low, resulting in a residual fatigue life ( $\approx 1700$  years) which is fair larger than the assumed reference life ( $L_{REF} = 100$  years). Namely, all design stress ranges ( $9.7 \div 22.3$  MPa) deriving from the passage of FLM4 heavy truck loads fall below the CAFL ( $\Delta\sigma_D = 0.737 \times 71 \text{ MPa} = 52.3 \text{ MPa}$ ).

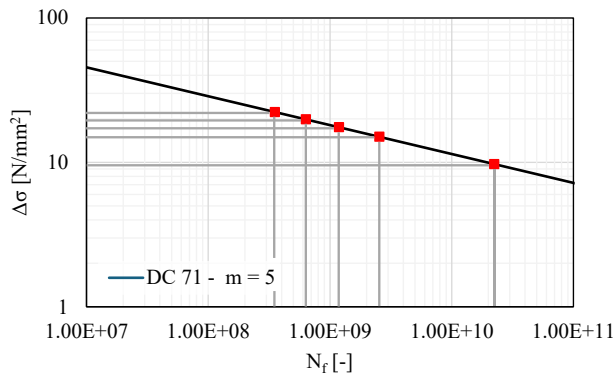
For the sake of comparison, the proposed probabilistic approach is hence applied in absence of degradation phenomena. The analytical approximation of the FLM4-based load spectrum is depicted in Figure 5.4-12. (reported values of  $\Delta\sigma_{max}$ ,  $\bar{n}$ ,  $p$  were estimated in order to minimize the discrepancy among the graphs).

Fatigue fragility FF for any value of  $0 \leq t \leq L_{REF} = 100$  years was hence estimated by assuming – in absence of more detailed information – a uniform temporal distribution of traffic loads through the years.

As expected, the minimum reliability level  $\beta_{MIN} = 19.2$  at  $t = L_{REF}$  is largely higher as respect to the EN1990 target  $\beta_{target} = 3.0$  (see Figure 5.4-13), and a residual fatigue life of about  $\approx 1300$  years is estimated.

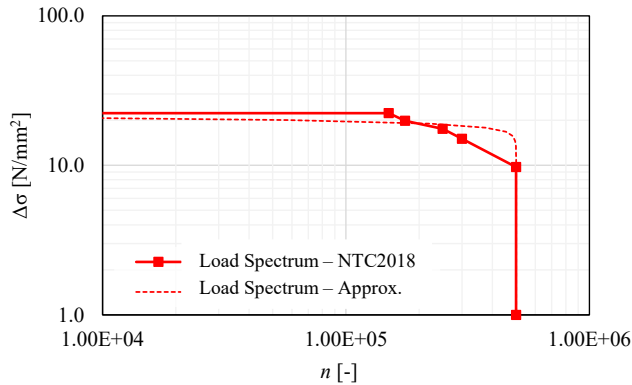
This is clearly in line with the deterministic approach, although a slightly higher conservativity is obtained through the proposed technique, i.e., most likely due to the more comprehensive accounting of uncertainty sources (e.g., fatigue strength, damage cumulation rule).

The critical corrosion level was hence estimated, i.e., the local corrosion degree  $\eta_{MAX,local}$  that would result in  $\beta_{min}(L_{REF}, \eta_{MAX,local}) = \beta_{target} = 3.0$ . It is worth remarking that, in absence of more detailed indications, the lowest exposure category C1 was considered to model corrosion evolution as per ISO 9224, i.e., due to the protective effect of weathering steel (see Figure 5.4-14, red curve).



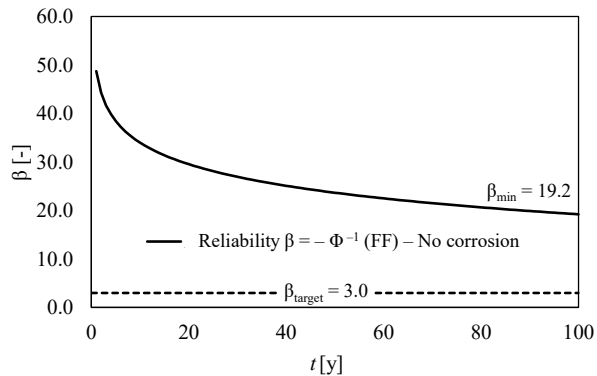
Total Fatigue Damage D **0.06** -  
Residual Life L<sub>RES</sub> 1695 years

Figure 5.4-11: Normative fatigue checks for most stressed welded details from Cadore bridge.



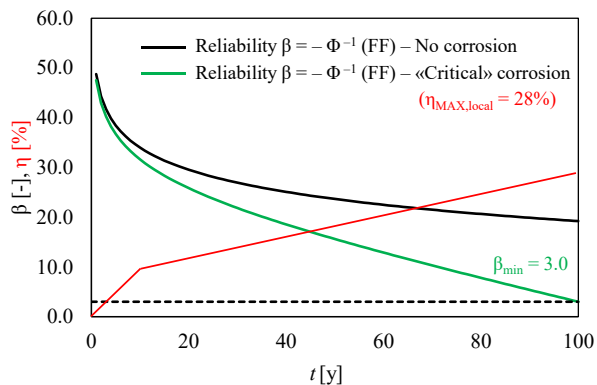
Maximum range  $\Delta\sigma_{MAX}$  22.3 N/mm²  
Cycles threshold  $\bar{n}$   $5 \times 10^7$  -  
Convexity parameter  $p$  17.1 -

Figure 5.4-12: Approximation of the load spectrum for the application of the probabilistic approach.



Minimum Reliability  $\beta_{min}$  **19.2** -  
Residual Life L<sub>RES</sub> 1255 years

Figure 5.4-13: Application of the probabilistic approach: Reliability trend for critical welded detail from Cadore bridge in absence of corrosive phenomena.



«Critical» corrosion  $\eta_{MAX}$  **28** %  
(localized, lower flange)

Figure 5.4-14: Application of the probabilistic approach: Estimation of the «critical» corrosion level for which  $\beta_{min} = \beta_{target}$  is obtained at the end of reference life.

## 6. Software tools for PM of RC and PSC bridges

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### 6.1 Underlying methodology for PM of RC and PSC bridges

Enhancing the structural and seismic safety of existing reinforced concrete (RC) bridges is a key objective for the civil engineering community. In Italy, most RC bridges were built before modern durability and seismic design standards, making them particularly vulnerable to deterioration—especially reinforcement corrosion. In this context, available national and international guidelines mainly rely on visual inspections and defect indices; however, these qualitative approaches would greatly benefit from analytical tools capable of quantitatively linking observed damage to reductions in structural capacity and, as a consequence, structural/seismic safety.

To address this need, a framework for the safety assessment and maintenance planning of existing RC and prestressed concrete (PSC) bridges is proposed. The methodology - described in detail in Deliverable 6.1a (DV6.1a) - employs visual inspection data as input to analytically evaluate local capacity reductions and to derive a global safety index. Corrosion propagation models are incorporated to characterise degradation over time, while uncertainties in corrosion extent and location are tackled through a parametric approach. The procedure allows evaluating degradation curves for safety index (i.e., capacity vs. demand ratio at Life Safety Limit State, LSLS) over time, together with a quantitative assessment of the safety evolution (trajectory). This output is deemed essential for informed decision-making on maintenance strategies.

Before presenting the key aspects relevant to the implementation of the procedure (see Section §6.2), the main features of the proposed methodology are briefly revisited in the following sections (more details are available in DV6.1). An illustrative application to an archetype RC bridge is then provided in Section §6.4.

#### 6.1.1 Recall of the framework for structural and seismic safety evaluation

The proposed methodology for assessing the structural and seismic safety of RC bridges over time is schematically illustrated in Figure 6.1-1. The procedure relies on a simplified yet mechanically-based approach for the structural performance assessment of existing RC bridges, also accounting for the effects of corrosion phenomena. The framework can be used to evaluate safety under either conventional traffic loads or seismic events, in line with the principles of performance-based seismic assessments and serviceability evaluations.

Each step of the procedure has been extensively discussed in the previous DV 6.1a. However, for the sake of clarity, a brief explanation is also provided in the next paragraphs. Details on the practical implementation of the procedure are then given in Section §6.2.

- *Data collection and defect indices assessment*

In the first fundamental step, a systematic collection of available structural information is performed. In this task, it is crucial to collect information on: (i) the geometric characteristics of the bridge, (ii) material mechanical properties, (iii) structural and reinforcement details, and (iv) any defect affecting the structure. This information can be obtained through historical design documentation and in-situ observations via visual inspections.

Focusing on bridge defects, the concept of “defect indices” can be adopted for this purpose, in line with the 2020 Italian Guidelines on Risk Classification and Management of Bridges. These indices are determined using standardized defect forms, where the severity and extent of different types of damage can be reported. Examples of possible defects include concrete cracking, spalling, and reinforcement corrosion. Specific defect forms are available for various structural components, such as abutments, piers, bearings, joints, columns (vertical supports), arches, beams (RC or PSC), and the deck.

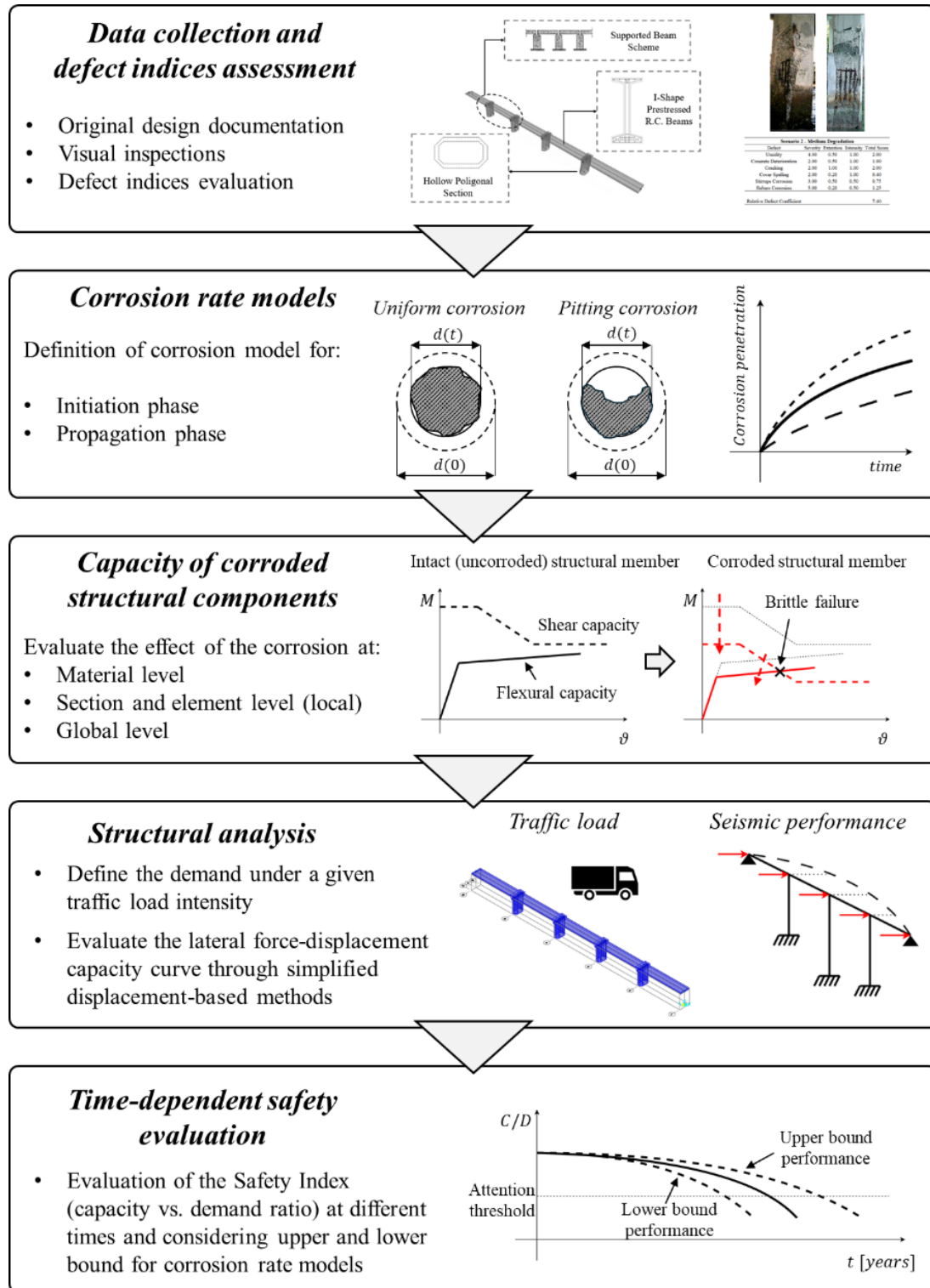


Figure 6.1-1: Proposed framework for time-dependent safety evaluation of existing RC bridges.

- *Corrosion rate model*

In the second step of the procedure, the corrosion scenario is defined and suitable corrosion-rate models are selected. These models are needed because, once the passive layer of steel is compromised by chlorides, the reinforcement enters the active corrosion (propagation) phase. Therefore, quantifying this degradation over time is essential to evaluate the reduction of structural capacity and safety and its trajectory.

Nowadays, it is widely acknowledged that corrosion can develop in different ways, thus requiring different modelling approaches:

- **Uniform corrosion:** occurs when damage spreads relatively uniformly along the reinforcement.
- **Pitting corrosion:** occurs when damage is concentrated in small critical zones, forming deep pits along the bar. This localised attack can severely reduce the local cross-section and bond capacity.

Both uniform and pitting corrosion are schematically illustrated in Figure 6.1-2.

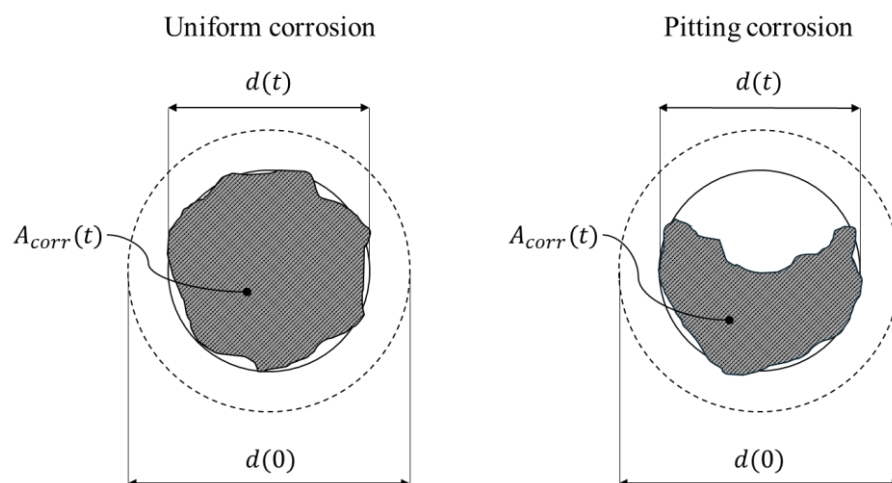


Figure 6.1-2: Schematic illustration of “uniform” and “pitting” corrosion.

Using corrosion-rate models allows the procedure to account for both the extent and severity of corrosion, providing a basis for predicting the evolution of safety over time. Concerning corrosion initiation, models based on Fick's second law can be used to evaluate chloride penetration into the concrete. On the other hand, for corrosion propagation, several corrosion rate models are available in the literature. An example of a comparison between different corrosion rate models is available in *Gentile et al. (2021)*. More information on corrosion rate models and their formulation is available in DV 6.1a.

- *Capacity of corroded structural components*

In the third step, the effect of corrosion on RC structural members is assessed. Corrosion phenomena can compromise the stiffness, strength, and ductility capacity of RC structural members, potentially leading to significant changes in their non-linear behaviour.

These effects arise from several interrelated mechanisms, including:

- Reduction of the steel reinforcement cross-sectional area;
- Modification of the steel stress-strain response;
- Cracking of the concrete cover due to the expansive action of corrosion;
- Degradation of the bond between steel and concrete;
- Deterioration of stirrups, leading to a loss of confinement in the concrete core.

Quantifying these mechanisms is fundamental to evaluate the reduction in terms of structural capacity of individual structural members and the overall structural safety.

In the proposed approach, the effect of corrosion in the section analysis is considered by changing the nonlinear behaviour of the construction materials, as a function of the rebars' mass loss,  $M_{loss}$  [%]. This approach leads to a modification of: *i*) steel reinforcement strength and deformation capacity; *ii*) concrete cover stiffness and strength; and *iii*) concrete core confinement (Figure 6.1-3).

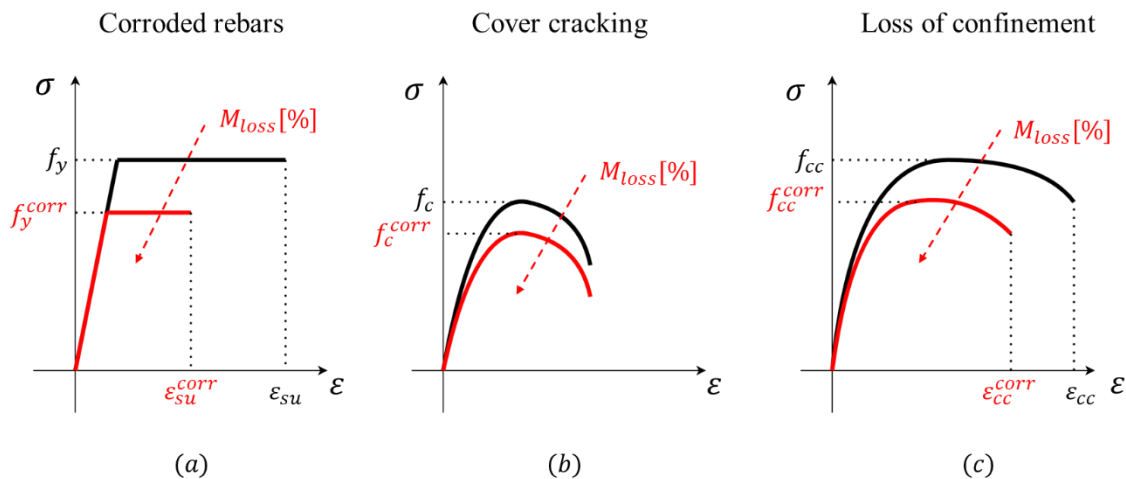


Figure 6.1-3. Qualitative example of the effects of corrosion on (a) steel rebars, (b) concrete cover and (c) concrete core.

More specifically, the mechanical properties of the reinforcement are updated according to the degradation laws proposed by *Imperatore et al. (2017)*, which provide experimentally derived relationships for both pitting and uniform corrosion. Whenever necessary, the compressive strength of the concrete cover is also reduced, according to the model proposed by *Coronelli & Gambarova (2004)*. Finally, for the concrete core, the loss of confinement is evaluated using the model proposed by *Mander et al. (1988)*. In this case, the effect of corrosion is considered by reducing the yielding strength of the stirrups. More details on the formulations are available in DV 6.1a, as well as in the cited papers.

The definition of stress–strain relationships for construction materials accounting for corrosion effect enables the assessment of flexural and shear capacities at both the section and member levels using classical structural design principles (Figure 6.1-4). To quantify the impact of corrosion on the strength and ductility of RC members, moment–curvature relationships and axial load–moment interaction diagrams are evaluated.

Intact (uncorroded) structural member

Corroded structural member

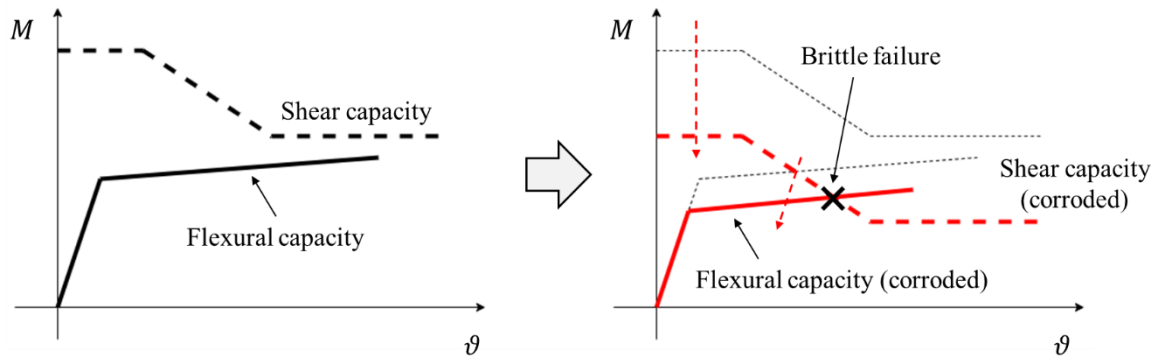


Figure 6.1-4. Example of modification of the hierarchy of strengths at the local component level due to the effects of corrosion.

- *Procedure for corroded PSC bridges*

As an integration of the previous DV 6.1a, the formulation for PSC bridges is discussed in more detail herein. For the PCS element, it is essential to consider two fundamental aspects that significantly influence the effects of corrosion phenomena. The first concerns the “chain effect”, related to the fact that each wire within the strand may undergo corrosion at a different rate, leading to non-uniform degradation. Consequently, the individual wires may fail at different times. Once the first wire ruptures, the tensile load is redistributed among the remaining intact wires, which in turn accelerates their stress increase and causes them to reach the failure condition earlier.

This aspect is accounted for in the model proposed by *Franceshini et al. (2023)*, in which a distinct stress–strain relationship is defined for each wire. Ultimately, all the wires are assembled into a single equivalent strand by means of an equivalent spring model, allowing the derivation of the overall stress–strain response of the entire strand. This simplified approach, known as the SCPS (Simplified Model for Corroded Prestressing Strands) method, is schematically illustrated in Figure 6.1-5.

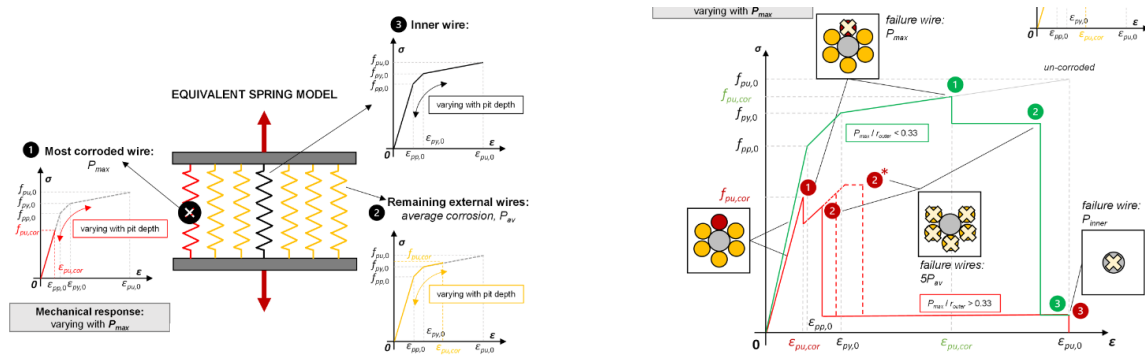
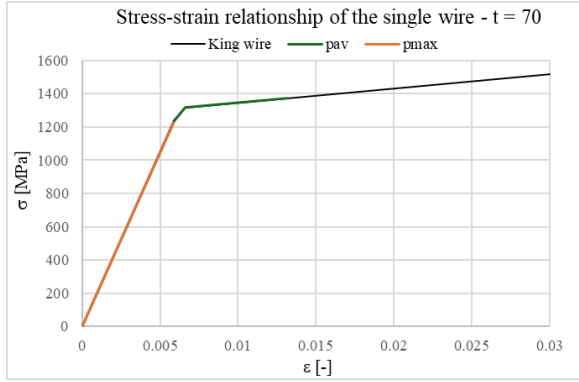
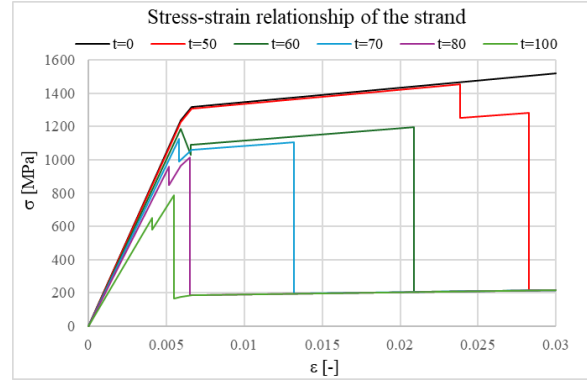


Figure 6.1-5: Schematic illustration of the SCPS method modified by *Franceshini et al. (2023)*.

As an example, Figure 6.1-6(a) shows different stress–strain relationships corresponding to each wire within the strand for a given value of  $p_{max}$ , while Figure 6.1-6(b) presents the resulting overall stress–strain relationship of the entire strand for different levels of  $p_{max}$ .



(a)



(b)

Figure 6.1-6: Example of stress-strain relationship obtained with the SCPS method for (a) individual wires, (b) the entire strand.

A further mechanism to be taken into account is the degradation of the steel–concrete bond, induced by crack propagation in the concrete and the presence of expansive corrosion products surrounding the strand. The principal consequences of this mechanism are the reduction of the effective prestressing force and the extension of the transfer length. These effects may lead to the premature failure of the structural element, often without exhibiting any evident warning signs.

Experimental investigations have shown that the transmission length  $L_{t,0}$  mainly depends on the concrete compressive strength ( $f_{cm}$ ), cover thickness ( $c$ ), equivalent strand diameter ( $\phi_{eq,0}$ ), and initial prestressing stress ( $\sigma_{p,0}$ ). Based on these parameters, *Anaya et al. (2022)* proposed the following simplified expression (Equation 37):

$$L_{t,0} = 30 \frac{c^{1/3} f_{cm}^{1/3} \phi_{eq,0}^{1/3}}{\sigma_{p,0}^{1/3}} \quad (37)$$

To account for corrosion effects, *Anaya et al. (2022)* proposed an analytical model calibrated against experimental data (Equation 38):

$$L_{t,corr} = k L_{t,0} \quad (38)$$

where  $k$  is a dimensionless coefficient defined as (Equation 39; Figure 6.1-7):

$$k = 1 + (\mu_{0.5} - \mu_{lim,Lt}) V_{det} \geq 1 \quad (39)$$

In Eq. (39),  $\mu_{0.5}$  is the section loss,  $\mu_{lim,Lt}$  is the limiting loss, and  $V_{det}$  depends on the difference between the first two terms and is calculated as (Equation 40-41):

$$V_{det} = 0.004 \frac{\sigma_{p,0} (f_{cm})^{1/9}}{(\phi_{eq,0})^{1/3} \left( \frac{c}{\phi_{eq,0}} \right)^3} \quad (40)$$

$$\mu_{lim,Lt} = 2250 \frac{(\phi_{eq,0})^{1/3} \left(\frac{c}{\phi_{eq,0}}\right)^2}{(\sigma_{p,0})^{5/4}} \quad (41)$$

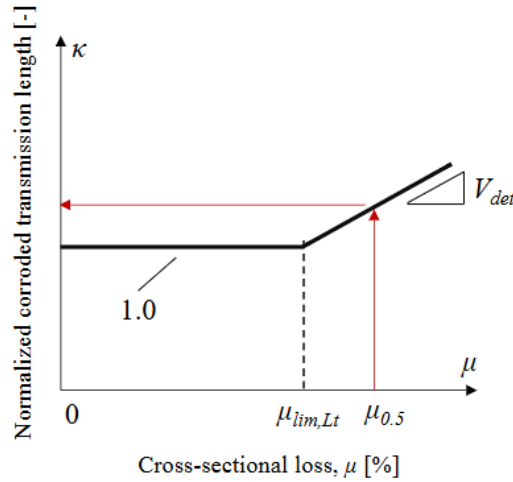


Figure 6.1-7: Variation of transmission length according to *Anaya et al. (2022)*.

The reduction of the effective prestressing stress due to corrosion is evaluated by correlating the loss of bond strength with the decrease in prestress. Based on the experimental results by *Wang et al. (2017)*, simplified analytical expressions are proposed to estimate the normalised residual bond strength and the corresponding loss of prestressing force.

- Structural analysis

Structural analysis is used to determine the demand on bridge components under traffic and seismic loads. For traffic, simplified methods - assuming transversely rigid decks and, for prestressed concrete bridges, negligible girder torsion - enable analytical evaluation of flexural and shear forces without resorting to numerical simulations. Traffic loads are defined according to bridge geometry, number of lanes, load positions, and lane schemes, in order to maximise internal demands on the structural components.

For seismic assessment, the global performance depends on the bridge's structural scheme. In simply supported bridges, piers are analysed independently, with the overall performance governed by the most critical element. In continuous bridges, longitudinal capacity is obtained by summing the capacities of individual piers under a uniform top displacement assumption, while transverse behaviour is evaluated either through displacement-based methods that account for modal contributions of each pier, or through simplified static-based approaches. More details can be found in DV 6.1a.

- Time-dependent safety evaluation

For the practical application of the proposed maintenance framework, safety is first quantified by comparing structural capacity with the demand, resulting in a Safety Index (Capacity vs. Demand ratio) that guides decisions on retrofitting, maintenance, or operational restrictions.

For traffic loads, demand is assessed using the design load models from the Italian Building Code (MIT, 2018) and compared with component capacities, accounting for corrosion effects. For seismic safety, the comparison is performed in the Acceleration Displacement Response Spectrum (ADRS), following the Capacity Spectrum Method (ATC, 1996) or the N2 method (Fajfar, 2000), as schematically shown in Figure 6.1-8.

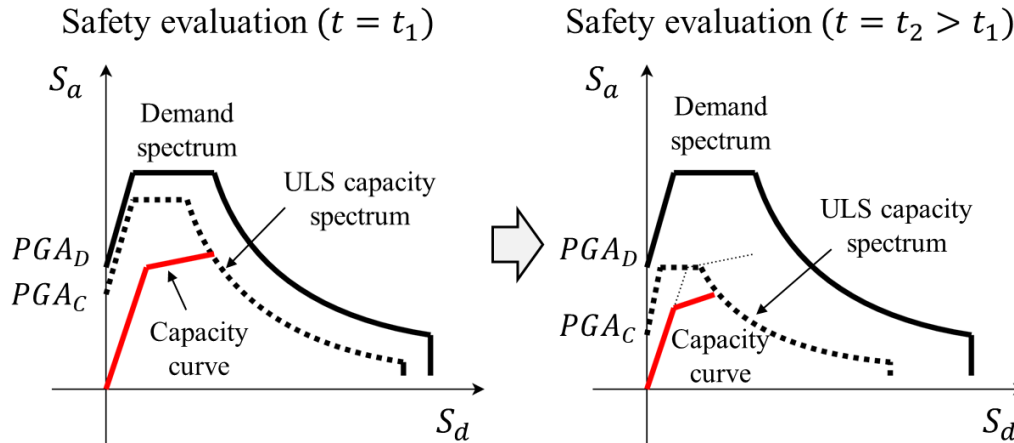


Figure 6.1-8: Example of time-dependent performance points evaluation at Ultimate Limit State (ULS) in the ADRS domain through the Capacity Spectrum Method.

By performing safety evaluations at different stages of the bridge's life, the reduction of the Safety Index over time is estimated (Figure 6.1-9). Corrosion-rate models are used to quantify the reduction in structural capacity due to degradation, allowing the assessment of safety deterioration for both seismic and traffic-related performance. When traffic forecasts are available, the predictive maintenance approach can also incorporate expected increases in traffic demand over time (e.g., Pugliese et al., 2022).

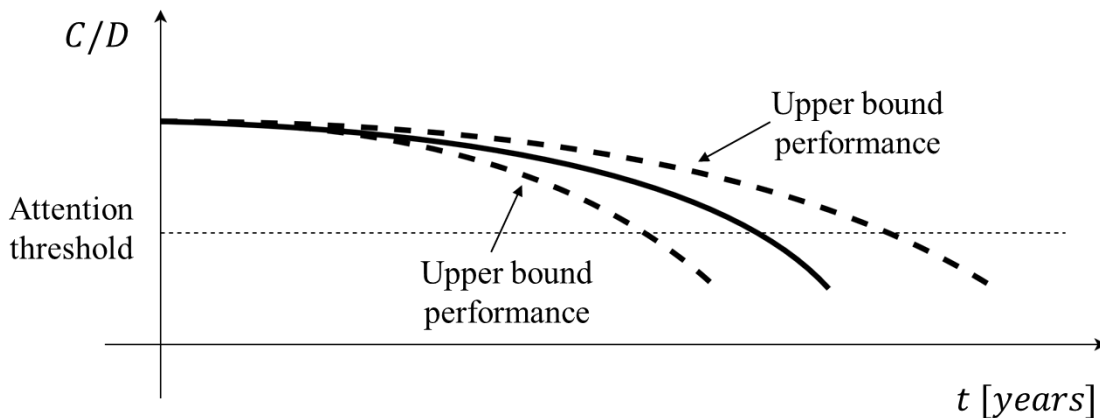


Figure 6.1-9: Schematic illustration of safety index reduction over time due to corrosion development.

## 6.2 Software tool for PM of RC bridges

The tool developed for assessing reinforcement corrosion degradation is structured into several independent modules, implemented through Python codes and supporting Excel worksheets. The main objective of the first module is to model the expected corrosion process in order to determine, in a coherent and quantitative manner, the onset of corrosion, the evolution of pitting, the initiation of cracking, and the eventual spalling of the concrete cover. This allows a consistent mapping between the qualitative defect index and the corresponding quantitative defect index.

- Input data

The analysis begins with the compilation of three JSON files that contain the geometric and mechanical input data. The file *section.json* includes the relevant geometric properties of the cross-section: section depth and width, concrete cover, bar diameter, stirrup diameter, stirrup spacing, coordinates of the reinforcement bars, and axial load. The file *steel.json* lists the mechanical properties of the reinforcing steel, including yield strength, ultimate strength, elastic modulus, and ultimate strain. Finally, the file *concrete.json* specifies the properties of concrete: characteristic compressive strength, shear modulus, and ultimate strain.

- Corrosion rate models

The first code, which deals with the modelling of the corrosion process, implements a comprehensive probabilistic framework that reproduces all phases of degradation: initiation of corrosion, pitting evolution, reduction of the effective steel area, formation and propagation of cracks, and eventual spalling of the concrete cover. The model relies on a Monte Carlo approach with 100,000 simulations, in which the main uncertain variables influencing the degradation process are sampled according to the probabilistic distributions prescribed by *DuraCrete (2000)* for the given exposure class.

Initially, the code imports the geometry from *section.json*, extracting in particular the concrete cover and bar diameter. These parameters are used to define several deterministic quantities, such as the reference time associated with concrete curing and the water–cement ratio of the mix. Probabilistic distributions are then assigned to all relevant variables, which are sampled to generate the full set of simulations. For each simulation, the initiation time of corrosion is computed, removing non-physical or unrealistic values. A lognormal distribution is subsequently fitted to the resulting dataset to obtain a theoretical representation of the simulated results. The script produces plots comparing the empirical histograms to the fitted theoretical curves and visualises the sampled distributions of the key variables, overlaying the corresponding theoretical probability density functions.

The model then computes the critical cover depth for the propagation phase using the relationship proposed by *Alonso et al. (1988)*, which depends on the ratio between concrete cover and nominal bar diameter. Based on this, the pitting factor is statistically characterised using a GEV distribution; its mean value is used to estimate the critical depth at which cracking initiates. In parallel, the initial corrosion current is calculated as a function of the water–cement ratio and the concrete cover. This parameter is essential for determining the time required for corrosion to reach the critical depth.

From this moment onward, the propagation phase is modelled. The corrosion current follows a time-dependent decay expressed through a negative power-law relationship, from which the corrosion

rate (mm/year of thickness loss) is derived. Time integration yields the pitting depth as a function of structural age, assuming no material loss prior to the initiation of corrosion. The pitting depth is then used to compute the reduction of the bar's effective area according to the model proposed by Val & Melcheres (1997), resulting in the complete temporal evolution of the residual steel area and of the percentage of corroded section. These results are exported in JSON format and illustrated through plots showing the loss of cross-section over time.

The subsequent step concerns the evolution of crack width in the concrete cover. Starting from the loss of steel area associated with the cracking initiation time, the model evaluates the residual cross-section in the following years. Using an empirical relationship that links the loss of area to crack width, the time-dependent evolution of crack opening is obtained. Before the initiation of cracking, the crack width is zero; afterwards, it increases progressively. These results are also exported as JSON files and represented graphically.

Finally, the time to spalling is identified as the first instant at which the crack width exceeds the threshold value of 1 mm. When this occurs, the corresponding point is highlighted in the crack-width evolution plot.

Overall, the model provides a complete and detailed representation of the corrosion-induced degradation process: from corrosion initiation to pitting evolution, from cracking to concrete spalling, including the progressive loss of steel cross-section. This probabilistic and time-dependent framework enables a robust quantitative assessment of the expected structural deterioration.

- Capacity of corroded structural components

The second analysis module focuses on evaluating the sectional capacity of reinforced concrete elements through a moment–curvature assessment that fully incorporates the degradation effects predicted by the corrosion model. This module is composed of two main codes: a master code that manages the workflow and data handling, and a dedicated *OpenSeesPy* (Zhu et al. 2018) routine that performs the fibre-section analysis. Its purpose is to quantify how corrosion-induced losses—affecting both the longitudinal reinforcement and the transverse confinement system—modify the flexural response of the section, from initial stiffness to yielding, ultimate capacity, and ductility.

The analysis begins by importing all required input data from a set of JSON files that mirror the structure of the previous module and ensure full consistency. These files contain the geometric characteristics of the cross-section, the mechanical properties of concrete and steel, and the degradation parameters generated by the corrosion model, such as bar area loss, crack width evolution, and the corresponding defect index expressed as *Mloss* levels. This separation between data and computation allows the procedure to be easily repeated for different structural configurations or deterioration scenarios, ensuring a robust and transparent workflow.

In the master code, it is possible to define different corrosion levels in terms of percentage mass loss, along with the specification of which sides of the section are affected. This allows the tool to capture asymmetric reductions in stiffness and strength, as well as shifts in the position of the confined core. For each defined *Mloss* value, the code updates the constitutive laws of both steel and concrete—confined and unconfined—according to the formulations presented in Section 6.1. Based on these updated material models, the sectional analysis is then recalculated using the corresponding degraded properties. In this way, the tool consistently reflects the influence of corrosion on the mechanical behaviour of the materials without explicitly modifying the cross-

sectional areas. The updated stress–strain relationships for concrete and steel are subsequently exported and plotted, providing a clear qualitative representation of how material performance evolves with increasing deterioration.

Once the section is fully defined, the code initialises the OpenSees environment and constructs a fibre section using Concrete04 for both confined and unconfined concrete regions and Steel01 for the longitudinal reinforcement. The cross-section is discretised using quadrilateral patches for the concrete core, cover, and voids, while the bars are represented through linear layers located at their actual coordinates. The moment–curvature analysis is executed through a simplified finite-element model consisting of two nodes connected by a zero-length section element. The base node is fixed, while the top node is subjected first to the axial load and then to an incremental rotation that induces curvature in the section.

The analysis proceeds incrementally until either divergence occurs or the ultimate deformation limits of the materials are reached. At each step, the resisting moment, curvature, and neutral-axis depth are recorded. This results in a complete moment–curvature curve that captures the transition from elastic behaviour to yielding and finally to the strength-degradation phase. The obtained curve is then automatically bilinearised using an energy-equivalence criterion, allowing the determination of the equivalent yield point, initial stiffness, ultimate moment, and post-yield curvature. These key parameters provide a condensed representation of the flexural capacity and ductility of the deteriorated section.

Once the moment–curvature analyses are completed for all deterioration levels, the code automatically exports the key results into an Excel file. This includes the full set of bilinearised parameters—yield moment and curvature, ultimate moment and curvature, and initial stiffness—as well as the complete moment–curvature curves when required. The Excel output is structured to facilitate direct integration with the subsequent calculation module, where the transformation from moment–curvature ( $M$ – $\kappa$ ) relationships to force–displacement ( $F$ – $\delta$ ) capacity curves is implemented through dedicated spreadsheets. By simply pasting the exported values into the corresponding input cells, the user can immediately obtain the pushover-based capacity representation of the section.

- Structural Analysis and time-safety evaluation

If the site-specific elastic response spectrum is also provided, the Excel tool proceeds to evaluate the seismic performance of the deteriorated section through the Capacity Spectrum Method. In this way, safety indices can be computed for each corrosion scenario, enabling a direct comparison of structural performance at different stages of degradation. Importantly, the Python codes also output the time associated with each analysed corrosion level, as predicted by the probabilistic degradation model. This allows the Excel tool to automatically generate the temporal evolution of the safety index, offering a clear and continuous representation of how corrosion progressively reduces the seismic capacity of the structure throughout its service life.

## 6.3 Integration with IT Platform at System Level

The results obtained through the developed software tool for PM of RC bridges can be potentially exported in a comma-separated values (.csv) file, in analogy with the tool for steel bridges previously discussed in Section 5.3.

The main data that can be exported in a .csv file are listed Table 6.3-1.

Table 6.3-1 – Outputs provided in the exported .csv file.

| DATA TYPE                  | FIELD                   | VALUE                         | DESCRIPTION                                    |
|----------------------------|-------------------------|-------------------------------|------------------------------------------------|
| General Information        | ID                      | 1, 2, ..., i, ..., n          | Label of the viaduct (Integer)                 |
|                            | Name                    | XXXXXXXXX                     | Name of the viaduct (String)                   |
|                            | Service Life            | XXX years                     | Reference service life as per EN1990 (Integer) |
|                            | Year of evaluation      | XXX years                     | Evaluation time for reported outputs (Integer) |
| Traffic Data               | Traffic category        | Local, Medium, Long           | cfr. Tab. 5.1.VIII NTC2018 (String)            |
|                            | AYT                     | 50'000 ÷ 200'000              | cfr. Tab. 5.1.X NTC2018 (Integer)              |
| Localization               | Lat                     | XXX                           | Latitude (Float)                               |
|                            | Long                    | XXX                           | Longitude (Float)                              |
|                            | Municipality            | XXXXXXXXX                     | Location (municipality) of the bridge (String) |
|                            | Region                  | XXXXXXXXX                     | Location (region) of the bridge (String)       |
| Structural Characteristics | Year Built              | XXX years                     | Construction period (Integer)                  |
|                            | Structural Type         | XXXXXXXXX                     | Structural typology (String)                   |
|                            | Primary Material        | XXXXXXXXX                     | Primary Material (String)                      |
|                            | Lenght                  | XXX m                         | Total length of the bridge (Float)             |
|                            | Max Span                | XXX m                         | Maximum Span Length (Float)                    |
|                            | N. of Spans             | XXX                           | Number of Spans (Integer)                      |
| Corrosion Data             | Corrosion Type          | XXXXXXXXX                     | Corrosion typology (String)                    |
|                            | Corrosion Rate          | XXX $\mu\text{m}/\text{year}$ | Corrosion rate (Float)                         |
|                            | Carbonation Depth       | XXX mm                        | Carbonation depth (Float)                      |
|                            | Chlorides at cover      | XXX %                         | Chloride concentration at cover (Float)        |
|                            | Secion Loss             | XXX %                         | Cross-section area loss (Float)                |
| Safety Index               | $\xi_V$ XXXyear         | XXX                           | Vertical Safety index at year XXX (Float)      |
|                            | $\xi_V$ XXXyear         | XXX                           | Vertical Safety index at year XXX (Float)      |
|                            | $\xi_V$ XXXyear         | XXX                           | Vertical Safety index at year XXX (Float)      |
|                            | $\xi_E$ XXXyear         | XXX                           | Seismic Safety index at year XXX (Float)       |
|                            | $\xi_E$ XXXyear         | XXX                           | Seismic Safety index at year XXX (Float)       |
|                            | $\xi_E$ XXXyear         | XXX                           | Seismic Safety index at year XXX (Float)       |
|                            | Critical threshold year | XXX-XXX year                  | Critical threshold year (Integer)              |

The above data can be easily imported into an IT, GIS-based platform to monitor the structural health of multiple CIs. These outcomes are deemed essential for informed decision-making on maintenance strategies. Overall, the framework can serve as a valuable decision-support tool to guide maintenance interventions, thus enhancing the long-term safety and resilience of existing RC bridge networks.

## 6.4 Illustrative Application

### 6.4.1 Description of the case-study RC bridge

As an illustrative application, the proposed procedure is applied to a case-study RC bridge. The selected case study is derived from the analysis of an available database of existing bridges located in Central Italy. This study enabled the organisation and classification of the viaducts according to three key aspects: (i) geometry (in terms of deck type and structural scheme), (ii) construction details, and (iii) material properties. Concerning the geometry, most viaducts consist of simply supported girder decks. The number of spans ranges from one to ten, with span lengths between 20–25 m and 35–40 m. For construction details, the most common pier type features a rectangular cross-section

equipped with steel bearings. Material properties were derived from in-situ investigations, providing values of concrete compressive strength and steel yield strength. Based on these data, a statistical analysis has been conducted to determine mean strength values: the concrete exhibited an average strength  $R_c = 42$  MPa, with a coefficient of variation of 0.31, while steel showed a yield strength  $f_{yk} = 450$  MPa, with a coefficient of variation of 0.13.

Based on these considerations, the selected case study consists of a girder-type viaduct, i.e. the most recurrent configuration in the database (Figure 6.4-1a). The structural scheme is a simply supported girder with a 30 m span, moment of inertia  $J_x = 50$  m<sup>4</sup>, and a linear weight of 150 kN/m. Vertical actions are transferred through steel bearings placed on concrete plinths measuring 5 m × 2.6 m × 7 m.

The piers have a regular configuration with a constant height of 10 m and a single-shaft hollow polygonal section with a net area  $A_c = 7$  m<sup>2</sup> (Figure 6.4-1b). The reinforcement is defined by a steel ratio  $\rho = 0.45\%$ , representing an intermediate condition among the reference structures. The mechanical properties adopted are the experimentally derived values  $R_c = 42$  MPa and  $f_{yk} = 450$  MPa. Longitudinal reinforcement consists of  $\phi 20$  bars, while transverse reinforcement is provided by  $\phi 10$  stirrups spaced at 25 cm.

The deck is composed of prestressed concrete I-girders fabricated using bonded wire technology (Figure Figure 6.4-1c). Each girder has a height of 1.6 m and a cross-sectional area of 0.45 m<sup>2</sup>, prestressed with 32 strands (each composed of 7 wires, diameter  $\phi = 0.6''$ ) arranged in three layers. Ordinary reinforcement includes 6  $\phi 14$  longitudinal bars, while shear resistance is provided by  $\phi 10$  stirrups at 20 cm spacing. The materials used correspond to concrete class C45/55 and steel B450C.

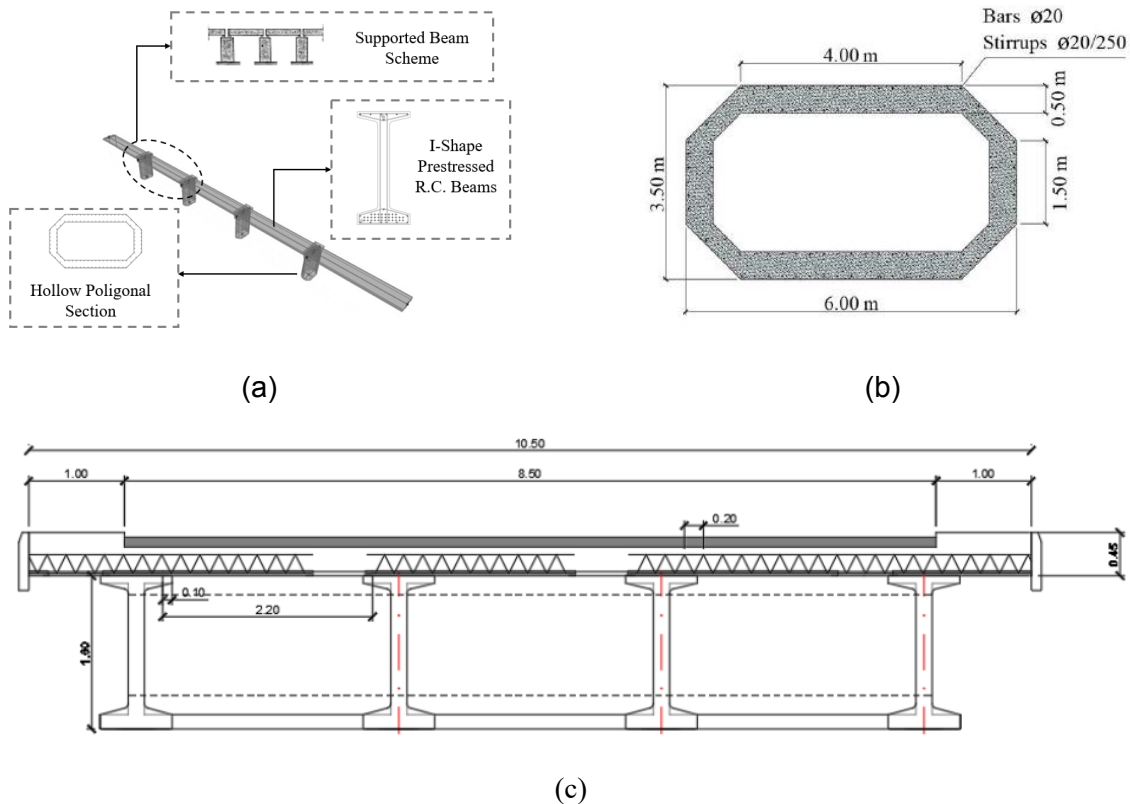


Figure 6.4-1. Geometric details of the case-study RC bridge: (a) structural characteristics of the bridge; (b) pier cross-section; (c) transverse section of the archetypal bridge

#### 6.4.2 Damage scenarios for individual components

As the case study is an archetype, no real damage scenarios are available. Therefore, hypothetical damage scenarios have been defined to demonstrate the procedure. Three levels of damage — low, medium, and high — are considered, each associated with a completed defect sheet, as illustrated in Figure 6.2-5. The corresponding defect indices are 3.6, 7.4, and 10.75, respectively.



| Scenario 1 - Low Degradation |          |           |           |             |
|------------------------------|----------|-----------|-----------|-------------|
| Defect                       | Severity | Extention | Intensity | Total Score |
| Umidity                      | 4.00     | 0.50      | 1.00      | 2.00        |
| Concrete Deterioration       | 2.00     | 0.50      | 1.00      | 1.00        |
| Cracking                     | 2.00     | 1.00      | 0.20      | 0.40        |
| Rebars Corrosion             | 5.00     | 0.20      | 0.20      | 0.20        |
| Relative Defect Coefficient  |          |           |           | 3.60        |

Figure 6.4-2. Degradation scenarios and example of defect sheet (after *Gentile et al., 2021*)

Consistently with the adopted corrosion-modelling framework, which provides an estimate of the pitting depth, each defect is associated with a percentage loss of reinforcement cross-sectional area. For the selected scenarios, these reductions are set to 5%, 10%, and 15%, respectively.

To account for the dependency of the corrosion effects on the location of the damaged section, additional position-based scenarios are also introduced. These configurations allow an assessment of how the structural response varies with the location of the pitting along the element, as illustrated in Figure 6.4-3. It should be noted that, in this deliverable, the procedure is applied specifically to a single pier, and therefore, the defined damage and position-based scenarios refer only to this pier.

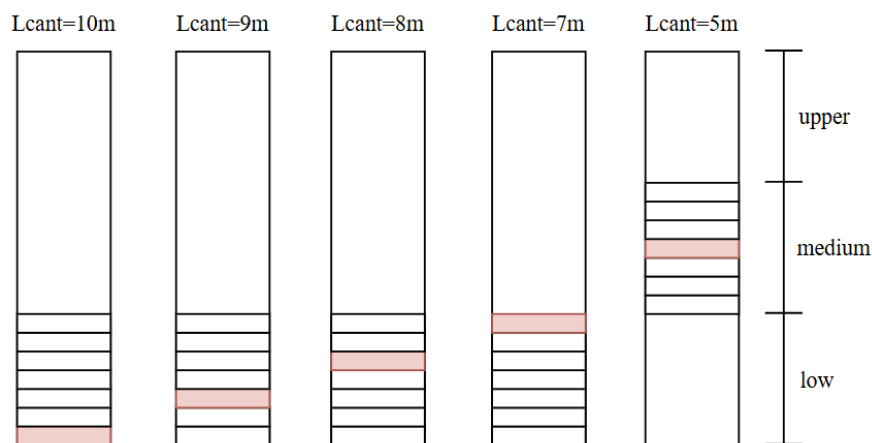


Figure 6.4-3. Scenarios of corroded section locations along the pier.

### 6.4.3 Pier capacity assessment

The flexural capacity of the pier, identified as the element most vulnerable to seismic action, was assessed at two levels:

1. **Section-level capacity:** the local flexural response was modelled through moment–curvature relationships using a fiber-based approach in OpenSees.py (Zhu *et al.* 2018). The section was divided into a confined core and an unconfined cover region, with longitudinal reinforcement represented by discrete steel fibers, as shown in Figure 6.4-4. For each material degradation state, the model updates concrete and steel properties and computes an equivalent bilinear curve, allowing the determination of yield moment, initial stiffness, and ultimate moment.

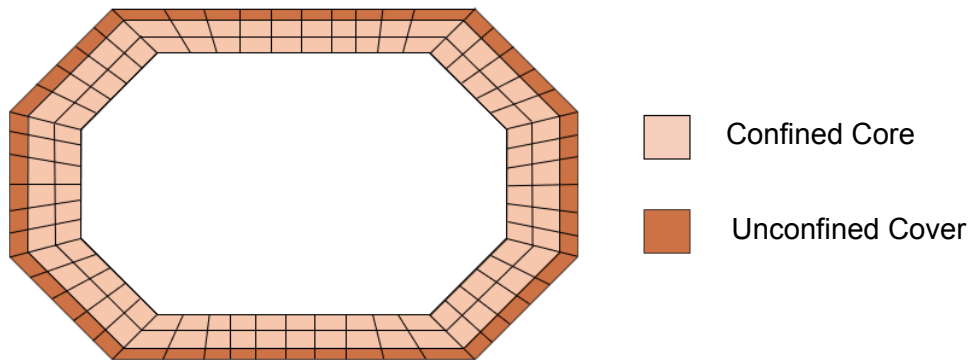


Figure 6.4-4. Fibre-based discretisation of the pier section.

2. **Element-level capacity:** the global force–displacement behaviour of the pier was derived from the section-level capacity and the equivalent cantilever length, through simple structural design principles. Shear strength was calculated by summing the contributions of concrete, transverse reinforcement, and axial load, adapting coefficients from both Italian and international codes.

Flexural and shear behaviours are interdependent. The comparison between flexural and shear capacity was carried out for both the pier base section and intermediate sections (Figure 6.4-5), highlighting that the interaction between bending and shear varies significantly depending on the location of the corroded section along the pier.

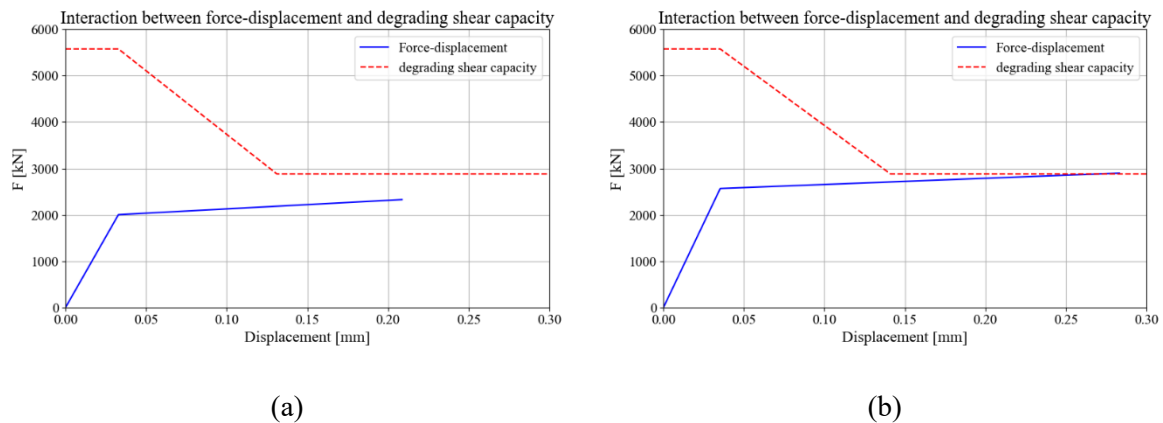


Figure 6.4-5. Flexural–shear interaction for (a) the corroded section at the pier base, (b) the corroded section located 8 m from the top

In general, it is assumed that a plastic hinge is most likely to form at the weakest section of the pier, which often corresponds to the location of the corrosion. However, the actual formation of the hinge must be assessed on a case-by-case basis, as it depends on the specific distribution of damage and loading conditions.

#### 6.4.4 Results and discussion

Figure 6.4-6(a) and Figure 6.4-6 (b) show, respectively, the moment–curvature relationships for a fixed plastic hinge location at the base, varying the corrosion level, and the moment–curvature curves for fixed corrosion levels of 10%, varying the location of the corroded section.

It can be noted that, as corrosion progresses, there is a gradual reduction in flexural strength and ductility, accompanied by a slight decrease in initial stiffness. This behaviour highlights the influence of cross-sectional loss due to corrosion on the nonlinear flexural response of the structural element.

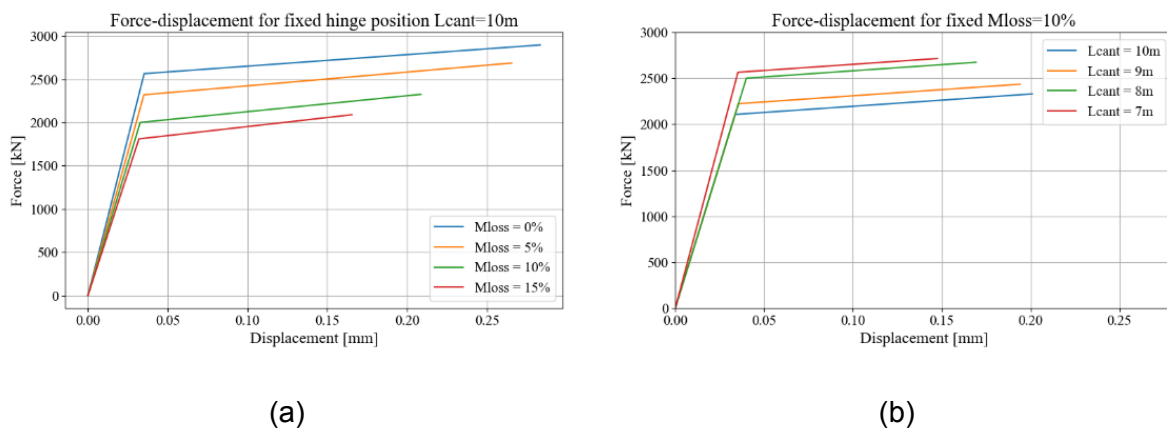


Figure 6.4-6. Moment–curvature relationships for (a) fixed cantilever length with varying degradation, (b) fixed degradation with varying cantilever length

Focusing on Figure 6.4-6(b), the minimum flexural capacity occurs when corrosion affects the base section. Conversely, moving the degraded zone to higher positions results in an increase in flexural strength, but with a reduction in overall displacement ductility, producing a more brittle failure mechanism. It is also observed that, for corroded sections located above approximately 7 m measured from the top of the pier, the section no longer governs plastic hinge formation, which tends to relocate back to the pier base. In other words, when the cantilever length is 8 or 9 m, the first plastic hinge still forms at the base.

Figure 6.4-7 illustrates the application of the Capacity Spectrum Method for three different degradation levels — (a) 0%, (b) 10%, and (c) 15% — considering the corroded section at the pier base. As expected, increasing the degradation leads to a safety reduction by almost 50%.

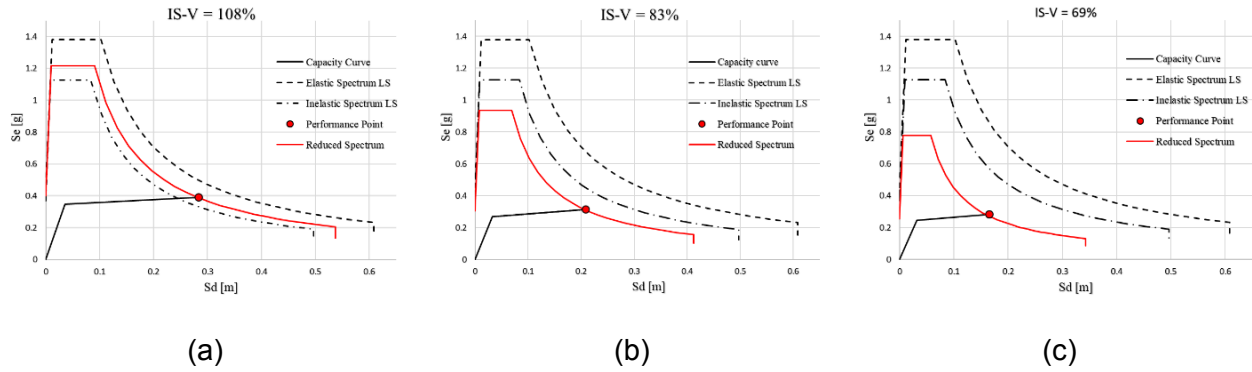


Figure 6.4-7. Capacity vs. demand evaluation and safety indices assessment for (a) 0%, (b) 10%, (c) 20% corrosion.

Finally, Figure 6.4-8 shows the time-dependent degradation of the safety index. The time-dependent analysis highlights how the location of corrosion significantly influences the evolution of structural safety. The curve corresponding to corrosion at the pier base begins to degrade earlier, but the reduction in seismic safety is more gradual. Conversely, when corrosion is located higher along the pier, degradation starts later but occurs more abruptly. This behaviour is attributed to the reduction in global ductility observed when the plastic hinge forms at a higher section. Consequently, a crossing of the curves over time is observed, suggesting that the defect location can significantly affect decision-making processes related to maintenance strategies. Alternatively, to account for variability in results due to the location of the corroded section, the pier safety curve can be represented as a fuse curve, as shown in Figure 6.4-8(b).

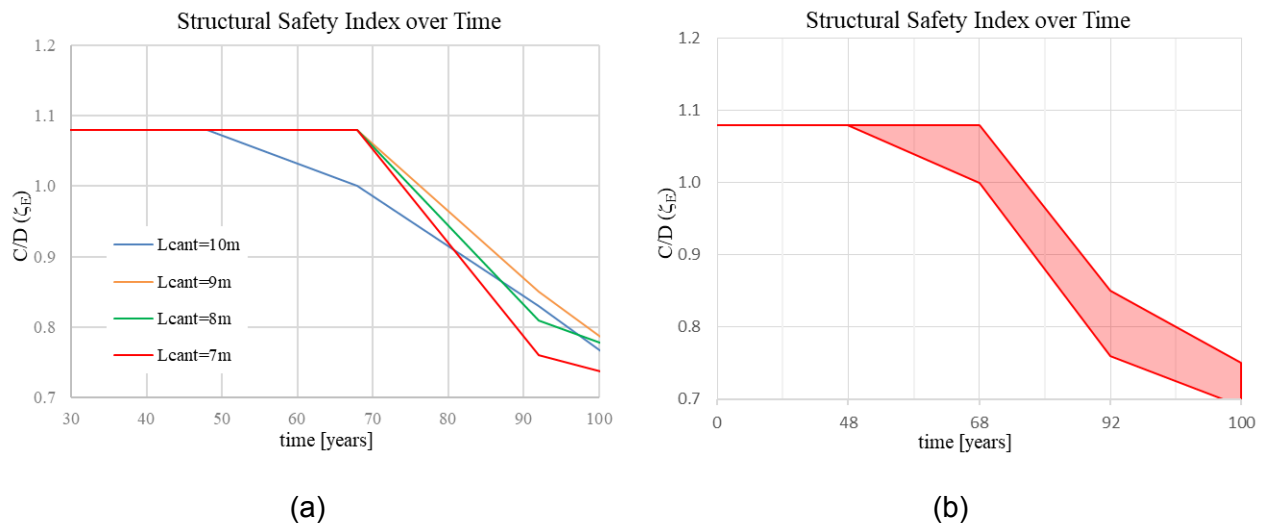


Figure 6.4-8. Structural Safety Index over time: results in terms of (a) individual curves and (b) range/domain of capacity over time.

## 7. Introduction to the IT Framework for Predictive Maintenance

### 7.1 Premise

The management of critical infrastructures, such as bridges, tunnels, hospital buildings, and industrial plants, requires advanced maintenance strategies to ensure safety, functionality, and long-term durability. The traditional paradigm of corrective or preventive maintenance is evolving towards a predictive approach based on continuous monitoring and advanced data analysis. The IT Framework presented here supports this transformation, enabling the integration of data from IoT sensors, predictive models, and higher-level processing, providing results accessible through customized interactive dashboards and decision support tools.

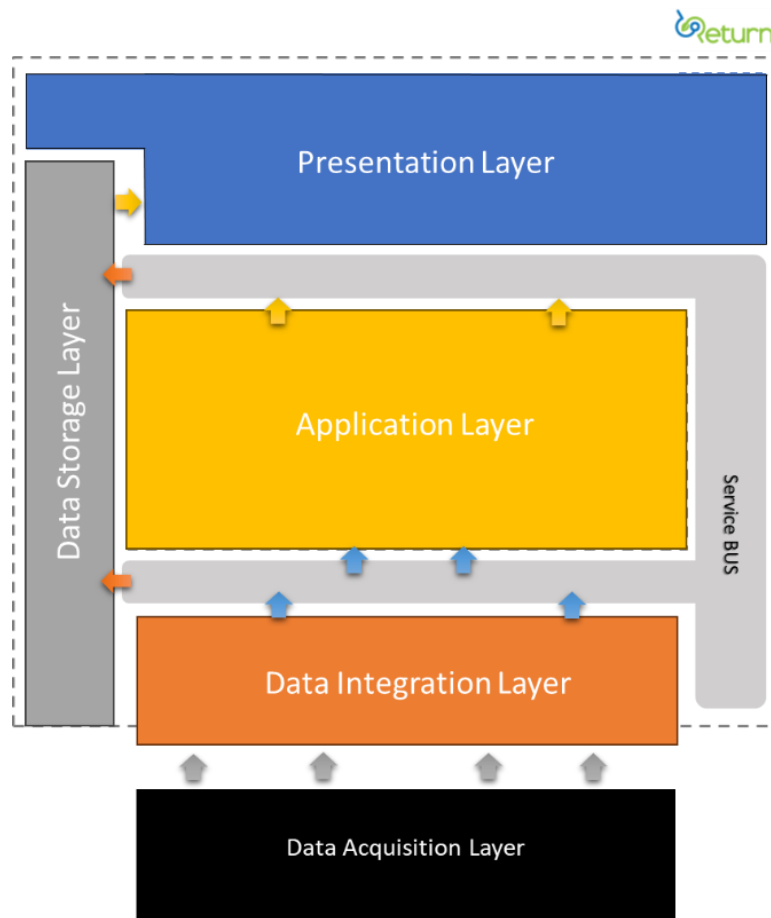


Figure 7.1-1: IT Framework Architecture

### 7.2 Architecture and Components of the Framework

The technological architecture of the framework is layered and modular, structured into four interconnected fundamental levels, which are detailed in the following subSections.

#### 7.2.1 Data Acquisition Level

The data acquisition level in the IT framework for predictive maintenance involves the collection of both real-time and non-real-time data from various types of sensors and monitoring systems.

Distributed sensors measure critical parameters such as vibrations, deformations, temperatures, energy consumption, and environmental conditions. These are complemented by advanced inspection systems, such as drones equipped with high-definition cameras and thermal cameras, connected via low-power IoT networks (LoRaWAN, NB-IoT) or high-capacity networks (5G), ensuring continuous connectivity and data transmission.

Alongside real-time monitoring, the framework also integrates data collected less frequently or in batch mode, typical of non-real-time monitoring systems. These data may come from periodic manual inspections, historical records stored in legacy systems, or devices that process and transmit aggregated data. Supporting this type of data enriches the analysis with less frequent but still relevant information for historical analysis, long-term predictive modeling, and contextualization.

In practice, the framework manages a continuous and multimodal flow of information, where real-time data enables timely anomaly detection and non-real-time data complete the picture with historical elements, qualitative data, or information from different sources. This dual acquisition mode is essential for effective and advanced predictive maintenance, capable of anticipating failures and optimizing interventions even over extended time horizons and complex assets.

In summary, the data acquisition level is designed to feed the system with heterogeneous raw data, both low-latency and delayed, providing a complete and integrated foundation for analysis, prediction, and advanced visualization processes.

### 7.2.2 Data Integration and Management Level

The data integration and management level in the IT framework for predictive maintenance leverages advanced ETL tools such as Node-RED, which plays a central role in data orchestration and pre-processing. Node-RED, with its visual flow interface, enables the creation of flexible and modular ETL pipelines to extract, transform, and load data from heterogeneous sources. Its event-driven nature makes it ideal for integrating raw data directly acquired from IoT sensors, electrical signals, or real-time monitoring systems, applying filters, conversions, and pre-processing algorithms to enhance the relevant physical quantities and reduce noise.

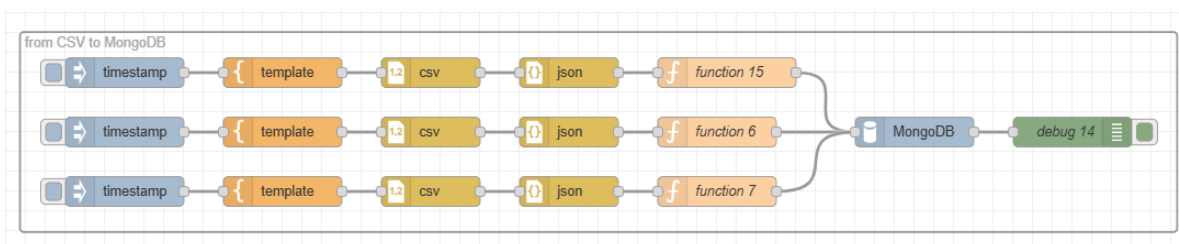


Figure 7.2-1: Node-RED flow for integrating CSV-based shared data, with ingestion and storage in the data lake.

At the same time, Node-RED manages processed and infrastructural data from legacy systems or scheduled batch ETL processes, integrating them into a coherent ecosystem. This includes historical data, contextual technical information, datasets from SCADA or Building Management Systems, allowing the harmonization and normalization of data to make them comparable and usable at upstream analytical levels.

The flexibility of custom nodes in Node-RED facilitates the implementation of validation logic, quality control, intelligent routing, and error handling, ensuring robustness and reliability of the data flow. Moreover, the platform supports connection to data lakes and data warehouses, feeding centralized

repositories with clean and structured datasets accompanied by metadata to ensure traceability, quality, and regulatory compliance.

This dynamic integration of raw, processed, and infrastructural data through Node-RED ensures a solid and flexible foundation for advanced analysis, predictive modeling, and interactive visualization, allowing the system to evolve over time alongside the monitored infrastructure and maintenance needs.

### 7.2.3 Processing and Analysis Level

The processing and analysis level of the IT framework for predictive maintenance is based on an advanced artificial intelligence and machine learning engine, which transforms normalized and historical data into operational knowledge. This engine integrates sophisticated anomaly detection algorithms, predictive models (such as regression, neural networks, ensemble models), and deep learning methods for pattern extraction, component remaining life estimation, and automatic recognition of structural defects.

A key aspect is the framework's ability to integrate ML models both internally and externally. Internal ML models are fed with clean, normalized data and operate in batch or real-time mode, providing predictive results and anomaly indicators. Simultaneously, the framework supports the import and use of results produced by external models, which may be specialized or based on different technologies. These results, integrated into the system, feed a higher-level processing stage that extracts higher-level information.

This higher-level processing uses environmental and infrastructural contexts, such as risk factors (hazards), infrastructure network configurations, and other contextual variables, to contextualize and enrich predictions. The synergy between internal and external models and the enrichment with contextual data enables the generation of more robust operational insights, translating technical-scientific forecasts into concrete operational guidance for optimizing maintenance strategies, planning interventions, and managing resources more effectively.

Furthermore, the use of digital twins – dynamic virtual representations of the real infrastructure – allows advanced simulations and real-time what-if analysis, supporting decisions based on sophisticated predictive scenarios.

In summary, this level provides an integrated cognitive system that does not merely produce predictive results but intelligently combines them with domain knowledge and contextual information to enable advanced and proactive predictive maintenance suitable for the complexity of critical infrastructures.

### 7.2.4 Visualization and Decision Support Level

The visualization and decision support level of the IT framework for predictive maintenance relies on interactive and scalable dashboards, developed with platforms such as Grafana and Kibana, which represent a key access point for all stakeholders. These dashboards allow the integration, visualization, and sharing of data from both direct acquisitions (raw data) and advanced processing (processed data or AI/ML model predictions).

Thanks to their modular architecture, Grafana and Kibana enable easy creation of customized widgets for each type of data or model added to the system. Operators can configure dynamic dashboards, filter data by time period or monitored component, drill down on specific anomalies, and

compare historical series. These tools also facilitate the integration of new data sources – whether real-time sensor streams, predictive model outputs, aggregated data, or historical records – maintaining scalability as infrastructure complexity grows.

Representing both raw and processed data addresses the needs of technical users (in-depth analysis, multi-parametric comparisons, degradation pattern study) and managerial users (KPIs, risk projections, operational summaries). Integrated alerting systems enable timely and contextualized notifications, with intelligent routing – for example, for critical issues on individual asset sections, escalation, or automatic reporting.

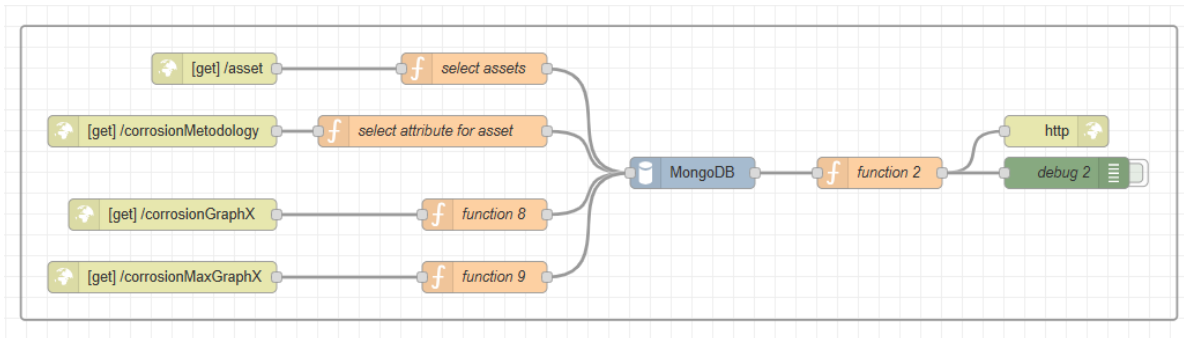


Figure 7.2-2: Node-RED flow for querying the data lake and delivering the retrieved information to the dashboard.

Finally, the ability to extend with new widgets and rapidly onboard new information or sources makes the visualization level a strategic asset for the entire architecture, ensuring adaptability and growth over time in response to evolving operational and analytical needs.

### 7.2.5 Integration of Raw Data, Models, and Results

The framework supports a complete flow from raw data acquisition to the presentation of results and advanced processing:

- ETL platforms such as Node-RED orchestrate data collection, transformation, and historical storage, managing heterogeneity in format, frequency, and criticality.
- Multi-level storage with MongoDB (semi-structured data and metadata) and MinIO (enabling dataset sharing and access via file system).
- The analytical layer uses normalized data to feed machine learning models operating both in batch and continuous inference, enabling real-time monitoring and prediction.
- Results and predictions are integrated into dashboards, allowing users to interact with current, historical, and projected data, facilitating informed decisions and effective planning.

This IT architecture represents a solid technological asset for adopting predictive maintenance in critical infrastructures, enabling proactive, efficient, and scalable asset management with a positive impact on safety, service continuity, and long-term economic sustainability.



Figure 7.2-3: Example dashboard displaying the acquired data and extracted information.

## 8. Conclusions

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In the present DV, data-based and analytic predictive maintenance methods (PMMs) for RC, PSC, steel and composite bridges – which were already discussed in DV6.1a – have been briefly recalled and implemented in the form of software tools.

Main features of developed software have been thoroughly described and some applicative examples have been shown.

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## Glossary

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**Predictive Maintenance (PM):** Data-driven strategy that monitors structural health in real-time (i.e., through SHM), and analyzes data with analytical techniques, AI and/or machine learning to forecast potential failures. It enables maintenance to be performed before a failure occurs, optimizing lifespan and minimizing unplanned downtime with respect to fix-when-broken approaches.

**Critical Infrastructures (CIs):** Point-wise or linear infrastructures whose performance critically affects the social, economical, environmental context of a (peri-)urban settlement. Examples include bridges, water and electricity distribution systems (linear CIs) or airports, power plants, waste treatment plants (point-wise CIs).

**Reliability (Index):** Level (Indicator,  $\beta$ ) of the structural safety within the framework of performance-based engineering. It is intrinsically connected to the probability of failure  $P_F$ , insofar as  $\beta$  can be mathematically interpreted as the number of standard deviations that separate the mean performance (or safety margin) of a system from its failure point.

**Fragility Curve:** Probabilistic tool depicting the probability of structural failure for a given system (e.g., a linear or point-wise CI) relative to a specific (multi-)hazard intensity measure (e.g., related to seismic events, fatigue, material degradation, etc.). It can be regarded as the foundation of quantitative risk analysis for structural purposes.

**IT Platform:** Information infrastructure which encloses all information regarding monitored data for the application of PM methods and enables an integrated visualization of results in support of decision making and infrastructure management.

**Bridge:** Linear infrastructure which is built over a river, another road, an orographic discontinuity to allow people and vehicles to cross from one side to the other. Bridges can be built by using multiple structural materials and technologies (e.g., RC, PSC, steel, composite steel-concrete, etc.).

**Structural Health Monitoring (SHM):** Automated, continuous or periodic process of implementing a damage detection strategies for CIs (such as bridges) on the basis of data collected from in-situ sensors and/or remote (e.g., space-borne) sensing.

**Material Degradation:** Deterioration of the physical, chemical, and mechanical properties of a structural material over time due to service use and environmental factors. If not properly prevented and/or recovered, material degradation can severely impair the structural performance of worn components. Relevant examples of material degradation include damp spots, oxidation, corrosion, cracking.

**Fatigue:** Progressive and localized mechanical damage that occurs when a structural component is subjected to cyclic, which leads to premature and sudden failure due to cracking initiation and propagation, i.e., even if the stress levels are below the material yield strength. It is typical of joints and connections in steel and composite bridges.

**Corrosion:** Electrochemical process of deterioration of materials such as metals, i.e., through reactions with their environment, such as oxygen, water, or acids. If not correctly prevented, corrosion results in a severe (and often localized) mass loss which can critically impair the performance of worn structural components.

**Service Life Extension:** Process of prolonging the service life of a CI beyond its original design lifespan through an ensemble of strategies such as maintenance, repairs, upgrades. When based on PM methods, it allows to maximize the infrastructure social, economical and environmental value, i.e., delaying replacement costs and improving sustainability.