# Structural capacity assessment of PC members subjected to different corrosion-induced damage scenarios

# Beatrice Belletti<sup>1</sup>, Simone Ravasini<sup>2</sup>

<sup>1</sup>University of Parma, Italy, beatrice.belletti@unipr.it

<sup>2</sup>University of Parma, Italy, simone.ravasini@unipr.it

# Abstract

In the past century, the prestressing construction technique has been widely used and nowadays several structures and infrastructures are beyond their service life. Mainly due to lack of maintenance, a significant part of existing bridges is subjected to deterioration phenomena induced by corrosion. In several countries, guidelines for bridges management propose defect-based index to provide a qualitative measure of bridge vulnerability. In this paper, the effect of different corrosion levels - detected in strands positioned at the bottom flange of I-shaped prestressed concrete bridge girders - is evaluated. According to practices adopted during visual inspections, corrosion levels are defined in terms of extension and intensity. In the paper, the dependency of the structural vulnerability on the position of the defect is also analysed.

# 1 Introduction

In this paper, the dependency of the flexural capacity of pre-tensioned concrete (PC) bridge girders on the corrosion of longitudinal prestressing strands, placed in the bottom flange of the cross section, is analysed. The exposure to wet and dry cycles of spray containing chlorides, due to the use of de-icing salts, is assumed, in this paper, the cause of the corrosion.

Structural vulnerability of bridges due to corrosive phenomena can be evaluated by adopting different levels of accuracy [1], [2]. For example, Italian Guidelines for the classification, assessment, and management of bridges (*GGLL*) [3] provide instructions for the vulnerability classification based on defect's survey, carried out during visual inspection and according to the so-called Level 1. Vulnerability assessment of each structural bridge component (such as beam, pile, etc.) is achieved in Level 2 by post-processing the data collected in specific sheets where the inspector must indicate the extension and the intensity levels of the noted defects. The classification of the level of deterioration reached in each component is finally obtained after combining the gravity attribute to the noted defects, together with their extension and intensity levels.

A specific sheet is available for prestressed beams and among the list of pre-defined defects: some of them are related to the initiation phase and others to the propagation phase of corrosion. Examples of defects related to the initiation phase are spots of moisture, water stagnations in box sections, etc., while examples of defects related to the propagation phase of longitudinal pre-tensioned reinforcement are longitudinal cracks in flanges, spalling of cover and depassivated wires, cross section loss of prestressing reinforcement, etc. It results that the same defect - that is in this paper related to the corrosion of pre-tensioned strands or wires placed in the bottom flange of the cross section - at different phases of propagation could be described, during the visual inspection, by noting different types of defects.

Higher accuracy in the measurement of the corrosion level can be achieved, after accurate inspections, useful also for the assessment of the bridge safety according to Level 4 and via numerical calculations. The knowledge level of the bridge can be identified after having collected as much as possible data from technical reports about the time of construction, the bridge geometry, drawing of reinforcement layout, mechanical properties, etc. Furthermore, the knowledge level of the defect, if related to corrosive phenomena, mainly depends on in-situ measurement related to the initiation phase (such as chloride content profile, carbonation depth, cover thickness) and propagation phase, such as electrochemical measurements (potential corrosion mapping,  $E_{corr}$ , corrosion rate,  $i_{corr}$ , resistivity,  $\rho$ ), surface damage mapping (opening widths of splitting cracks, *w*, cover spalling), cross section loss, pith depth, etc.. The knowledge of these latter data allows to apply the most recent modelling techniques available in literature to estimate the effects of corrosion in strands and their residual tensile capacity [4], [5], [6], [7].

In this paper, the corrosion in longitudinal pre-tensioned strands during the propagation phase is defined by a single defect. The intensity level,  $k_{int}$ , and the extension level,  $k_{ext}$ , of the defect are calibrated, based on the surface damage that can be observed during visual inspection, such as splitting cracks, cover spalling and section loss of wires. The role played by the corrosion intensity,  $k_{int}$ , of the defect is evaluated with two different approaches based: (i) on visual inspection with a qualitative estimation, (ii) on the most recent models available in literature, to account quantitatively the tensile capacity of corroded strands. The capacity of bridge girder is evaluated by adopting a probabilistic approach via Latin Hypercube sampling technique [8]. A Matlab code, still used in [9], was adopted for the purpose. The aleatory uncertainties related to mechanical properties are considered in the probabilistic assessment, while the geometrical features are treated as deterministic values.

The proposed procedure allows to assign a severity to the defect-based index after visual inspections – according to Level 1 and Level 2 - based on the same engineering criteria adopted during numerical assessment - in Level 4 - to account for the effects of corrosion. For this reason, the proposed defect-based index could also be adopted for a simplified calculation of the resistant moment of *PC* bridge girders subjected to corrosion of longitudinal strands in the bottom flange and characterised by flexural failure due to strands' rupture.

# 2 Case study: bridge girder

# 2.1 Geometrical features

The selected case study is a viaduct, designed in 1976 and executed in 1988, [10].



Fig. 1 (a) Longitudinal and (b) transversal section of the deck.



Fig. 2 Edge bridge girder details: (a) longitudinal prestressing steel reinforcement, (b) normal and prestressing steel reinforcement at sections A-A and B-B.

The analysed isostatic deck is realised with four simply supported precast prestressed concrete girders and poured in place slab 0.2 m thick. The length of the deck is 34.4 m, and the width is 14.25 m, Fig. 1. The four transverse beams are made of prefabricated prestressed concrete. Fig. 1 shows the geometrical features of the deck and the edge bridge girder analysed in this paper. The I-shaped precast beams are prestressed with fully embedded pretensioned 0.6" strands with deviated tendons, Fig. 2. Since 18 tendons are deviated, Fig. 2 shows that at section A-A (having distance equal to L/8 from the support) and at section B-B (at midspan), respectively, 26 and 44 strands are placed in the bottom flange of the cross section. The effects of the corrosion in deviated tendons - placed in the web on the moment and shear resistance in Section A-A will be analysed in future works by Authors. Indeed, corrosion in deviated tendons – according to *GGLL* - is noted, during visual inspections carried out in Level 1, with a different type of defect than corrosion of longitudinal strands in the bottom flange.

The reinforcement adopted in the slab, the longitudinal view of the normal steel reinforcement and the reinforcement layout of cross section at midspan and extremities of the beams are also shown in Fig. 2.

# 2.2 Mechanical properties

The characteristic mechanical properties are known for the case study [10]. The characteristic value of cube compressive strength of concrete adopted for the precast girders,  $f_{ck,cube}$ , is equal to 45 N/mm<sup>2</sup>, while for the concrete slab is equal to 35 N/mm<sup>2</sup>. Cylinder compressive strength of concrete,  $f_c$ , is calculated by multiplying the cube compressive strength,  $f_{ck,cube}$ , times 0.83.

The characteristic value of 0.1% proof strength of prestressing steel,  $f_{p0.1k}$ , is equal to 1500 N/mm<sup>2</sup>; while the characteristic value of tensile strength of prestressing steel,  $f_{ptk}$ , is equal to 1860 N/mm<sup>2</sup>. The mean value of tensile strength of prestressing steel is calculated from the characteristic value according to the relation reported in *JCSS* [11] ( $f_{pt} = f_{ptk} + 66$ ).

FeB 44K steel is adopted for mild reinforcement having characteristic value of yield strength in tension,  $f_{yk}$ , equal to 430 N/mm<sup>2</sup> and characteristic value of tensile strength,  $f_{tk}$ , equal to 540 N/mm<sup>2</sup>. The mean value of the yield strength,  $f_{y}$ , is considered equal to 451 N/mm<sup>2</sup>, as also reported also by Miluccio et al. [12], while the ultimate strain,  $\varepsilon_u$ , is obtained from *STIL* software [13]. The data for steel are relative to prestressed concrete bridge girders constructed between 1970 and 1980. The ratio  $f_u f_y$  is kept constant and equal to 1.25. Table 1 summarizes the random variables related to the material properties and the adopted statistical distributions.

Material	Var.	Units	Char. values	Bias [-]	CoV [%]	Stand. dev. σ	Mean values	Distr.	Ref.
Concrete beam	$f_c$	[MPa]	37.35	-	11.40	-	45.35	Logn	[12]
Concrete deck slab	$f_c$	[MPa]	29.05	ŀ	11.40	-	37.05	Logn	[12]
Mild steel	$f_{y}$	[MPa]	430.00	1.05	-	-	451.00	Logn	[12], [13]
	$\mathcal{E}_{u}$	[‰]	135.00	1.00	-	3.50	135.00	Logn	[12], [13]
Prestressing steel	$f_{pt}$	[MPa]	1860.00	1	-	40.00	1926.00	Norm	[11], [14]
	$\varepsilon_{pu}$	[‰]	50.00	1.00	-	3.50	50.00	Norm	[11], [14]
	$\sigma_{p0}$	[MPa]	-	-	12.00	-	966.18	Unif.	[12]
*The bias is the ratio between the mean and characteristic values, Caspeele & Van Den Hende [15]									

Table 1 Random variables related to the material properties and statistical distributions.

The applied prestress value,  $\sigma_{p0} = 966.18$  MPa, is estimated by considering immediate and long-term losses. A uniform distribution with a coefficient of variation of 12 % is considered [12], Table 1.

# 3 The SCPS model for corroded strands

### 3.1 The corrosion morphology of corroded strands

Nowadays, emerging techniques for the measurement of the corrosion morphology are under development that could have a great impact on the inspection procedures in future years. Among them, 3D-scanning of damage surface in concrete and in reinforcement is considered by Authors an innovative technique that could allow to measure the damage defects on site without retrieving the reinforcement from members. For example, the corrosion morphology of corroded strands could be described by pith depths in wires [7], [16] directly measured on site.

Since the bond and the residual prestressing force could be strongly affected by concrete cover removal, required for scanning the corrosion morphology, few measurements in limited zones can be planned during inspections of prestressed members with pre-tensioned reinforcement. For this reason, laboratory measurements on naturally corroded reinforcement are extremely useful to provide statistical distribution to be updated with the few data that can be collected from in-situ measurements [7], [17].

Also to this aim, Authors edited a Corroded Prestressing Database (<u>https://www.fib-international.org/commissions/databases.html</u>) [18] to provide extensive data on the corrosion type, the geometry, the mechanical properties, the corrosion morphology and the mechanical behaviour of corroded prestressing reinforcement. The collection of data is still ongoing and new data, obtained also with *X*-ray tomography, will be soon added to the database [19]. Authors invite providers to add Findable, Accessible, Interoperable, and Reproducible (*FAIR*) data to this Open Access database.

Fig. 3(a) shows the 3D-scan of corroded strands, carried out at the University of Parma, and the pith depth measurement of the external wires executed with Zeiss Inspect software. An example of post-processing data, resulting from 3D-scan of 0.5" seven-wire naturally corroded strands (having outer wires' diameter,  $r_{out}$ , equal to 4.26 mm and the inner one,  $r_{in}$ , equal to 4.38 mm) and retrieved from beams, exposed to wetting and drying cycles with see water, are presented in Fig. 3(b). The probabilistic distribution of the corrosion morphology is obtained from the 3D-scan as described in following steps, [19], [20]:

- (i) The 3D-scan is performed, Fig. 3(a), and for each strand, several cross sections 10 mm spaced are identified, Fig. 3(b),
- (ii) At each *j*-th cross section 10 mm spaced the maximum pit depth of each external wire,  $P_{max\_w,i}$ , is measured, Fig. 3(c), and the maximum pit depth of the most corroded wire at the *j*-th section is identified as  $P_{max\_sect,j} = max(P_{max\_w,i})_{i=1,6}$  [19], Fig. 3(d). The ratio between the maximum pit depth of the most corroded wire and the radius of the external wire,  $P_{max\_sect,j}/r_{out}$  is the input parameter of the *SCPS* model (illustrated in Paragraph 3.2), for the assessment of the tensile capacity of corroded strands at the *j*-th cross section,
- (iii) The maximum value of the variable  $P_{max\_sect,j}/r_{out}$ , measured along the entire length of the strand, is calculated as  $P_{max,p}/r_{out} = \max(P_{max\_sect,j})/r_{out}$ ,
- (iv) The probability density function, *PDF*, of the variable  $P_{max\_sect,j}/r_{out}$ , is plotted, Fig. 3(e),
- (v) The mean value,  $\mu(P_{max\_sect,j})/r_{out}$ , and the standard deviation value,  $\sigma(P_{max\_sect,j})/r_{out}$ , of the probability density function of the variable  $P_{max\_sect,j}/r_{out}$  are calculated, Fig. 3(f)-(g),
- (vi) A correlation between the mean value,  $\mu(P_{max\_sect,j})/r_{out}$ , and the maximum measured value,  $P_{max_p}/r_{out}$  is established by fitting the measured data of the scanned strands, as given in the Eq. (1):

$$\frac{\mu(P_{\max\_sect,j})}{r_{out}} = 0.4113 \cdot \frac{max(P_{\max\_sect,j})}{r_{out}}$$
(1)

(vii) A correlation between the standard deviation value,  $\sigma(P_{max\_sect,j})/r_{out}$ , and the maximum measured value,  $P_{max,p}/r_{out}$  is established by fitting the measured data of the scanned strands, as given in the Eq. (2):

$$\frac{\sigma(P_{\max\_sect,j})}{r_{out}} = 0.2238 \cdot \frac{max(P_{\max\_sect,j})}{r_{out}}$$
(2)

To simplify the description of the corrosion morphology, the *SCPS* model assumes, at the *j*-th section, the same pit depth for the remaining five outer wires,  $P_{av\_sect,j}/r_{out}$ , Fig. 3(d), that is calculated as a function of the pith depth of the most corroded wire,  $P_{max\_sect,j}/r_{out}$ , as given in Eq. (3) [19]:

$$\frac{P_{av\_sect,j}}{r_{out}} = 0.378 \left(\frac{P_{\max\_sect,j}}{r_{out}}\right)^2 + 0.25 \frac{P_{\max\_sect,j}}{r_{out}}$$
(3)

The 3D-scan of the section loss of each wire,  $A_{loss\_w,i}$ , allows to directly relate the maximum pit depth of each wire,  $P_{max\_w,i}$ , with the residual cross-sectional area of the corroded wires,  $A_{res\_w} = A_{w,i} - A_{loss\_w,i}$ , Fig. 3(c), being  $A_{w,i}$  the cross section of the uncorroded wire.



Fig. 3 Measurement of the corrosion morphology: (a) 3D-scan of the strand, (b) analysis of the data at each *j*-th section, (c) measure of the maximum pit depth of each external wire,  $P_{max\_w,i}$ , (d) definition at each *j*-th section of the pit depth of the most corroded wire,  $P_{max\_sect,j}/r_{out}$ , (e) *PDF* of the variable  $P_{max\_sect,j}/r_{out}$ , (f)-(g) mean value,  $\mu(P_{max\_sect,j})/r_{out}$ , and standard deviation value,  $\sigma(P_{max\_sect,j})/r_{out}$ , as function of the maximum measured value,  $P_{max\_p}/r_{out} = \max(P_{max\_sect,j}/r_{out})$ .



Fig. 4 Relation between the residual area of outer wires,  $A_{res w,i}/A_{w,i}$ , vs  $P_{max w,i}/r_{out}$  [16], [21].

Fig. 4 shows that the fitting of the measured data allows to establish a relation between the normalised remaining area,  $A_{res_w,i}/A_{w,i}$ , and the pit depth of each wire,  $P_{max_w,i}$ , as given in Eq. (4):

$$\frac{A_{res\_w,i}}{A_{w,i}} = \left(1 - 0.33 \cdot \frac{P_{\max\_w,i}}{r_{out}}\right) \quad \text{for } \frac{P_{\max\_w,i}}{r_{out}} \le 0.33 \tag{4}a$$

$$\frac{A_{res\_w,i}}{A_{w,i}} = \left[0.9 - 0.539 \cdot \left(\frac{P_{\max\_w,i}}{r_{out}} - 0.33\right)\right] \quad \text{for } \frac{P_{\max\_w,i}}{r_{out}} > 0.33 \tag{4}$$

### 3.2 The formulations of the SCPS model

The tensile resistance of corroded strands can be evaluated by assuming that wires behave as springs working in parallel [22], refer to Fig. 5(a). The tensile resistance of each wire can be calculated by multiplying the residual cross-sectional area,  $A_{res_w}$ , times the strength of the steel detected on the uncorroded stress-strain diagram in correspondence of ultimate strain value,  $\varepsilon_{pu,corr}$ , of the corroded wire [16]. According to the *SCPS* model, that states for Simplified model for Corroded Prestressing Steel, both residual cross-sectional area,  $A_{res_w}$ , and the ultimate strain value,  $\varepsilon_{pu,corr}$ , of corroded wires are only dependent on the variable,  $P_{max\_sect,j}/r_{out}$ , it results that the pit depth of the most corroded wire at *j*-th section, is the unique input parameter required for the calculation of the tensile capacity of the strand at *j*-th section, see Fig. 5. The stress-strain diagram adopted by the *SCPS* model [16] is a trilinear relationship, as given in Eqs. (5) (Fig. 5(b)), where  $f_{pp} = 0.7 f_{pt}$  and  $f_{py} = 0.882 f_{pt}$ , while  $\varepsilon_{pp} = f_{pp}/E_p$  and  $\varepsilon_{py} = 0.01$ :

$$\sigma = \begin{cases} E_p \cdot \varepsilon & \varepsilon \leq \varepsilon_{pp} \\ f_{pp} + E'_p \cdot (\varepsilon - \varepsilon_{pp}) & \varepsilon_{pp} < \varepsilon \leq \varepsilon_{py} \\ f_{pp} + E''_p \cdot (\varepsilon - \varepsilon_{py}) & \varepsilon_{py} < \varepsilon \leq \varepsilon_{pu} \end{cases} \qquad E''_p = \frac{f_{py} - f_{pp}}{\varepsilon_{py} - \varepsilon_{pp}}$$
(5)

The relation between the pith depth of the wire,  $P_{max\_w,i}/r_{out}$ , and the ultimate strain of the corroded wire,  $\varepsilon_{pu,corr}$ , is given in Eq. (6) and shown in Fig. 5(c). If  $P_{max\_w,i}/r_{out} > 0.33$ , the hardening branch (in the range  $\varepsilon_{py} - \varepsilon_{pu}$ ) disappears; if  $P_{max\_w,i}/r_{out} > 0.86$ , the rupture of the wire occurs before reaching the elastic strain  $\varepsilon_{pp} = f_{pp}/E_p$ . Since prestress value ranges around 0.5 – 0.6  $f_{pl}$ , it results that, for corrosion levels characterised by pit depth  $P_{max\_w,i}/r_{out} > 0.86$ , it becomes critical for the corroded wire to sustain the prestressing force and the dead weight of the bridge girder, only.

$$\frac{\varepsilon_{pu,corr}\left(\frac{P_{\max\_w,i}}{r_{out}}\right) - \varepsilon_{py}}{\varepsilon_{pu} - \varepsilon_{py}} = 1 - 3.03 \cdot \frac{P_{\max\_w,i}}{r_{out}} \quad \text{for } \frac{P_{\max\_w,i}}{r_{out}} \le 0.33$$

$$\frac{\varepsilon_{pu,corr}\left(\frac{P_{\max\_w,i}}{r_{out}}\right)}{\varepsilon_{pv}} = 1 - 0.599 \cdot \left(\frac{P_{\max\_w,i}}{r_{out}} - 0.33\right) \quad \text{for } \frac{P_{\max\_w,i}}{r_{out}} > 0.33$$
(6)a
(6)b

Since at the *j*-th section, the same pit depth for the remaining five outer wires,  $P_{av\_sect,j}/r_{out}$ , Fig. 3(d), is calculated as a function of the pith depth of the most corroded wire,  $P_{max\_sect,j}/r_{out}$ , it results that the relation tensile resistance vs strain of the strand, instead of being characterised by 7 softening steps corresponding to the sequential failure of the 7 wires, is characterised by 3 steps corresponding to:

- (i) The failure of the most corroded wire (point 1 in Fig. 6(a)), Eq. (7)a,
- (ii) The failure of the remaining five outer wires (point 2 in Fig. 6 (a)), Eq. (7)b,
- (iii) The failure of the inner wire (point 3 in Fig. 6 (a)), Eq. (7)c.

The Eqs. (7) describe the tensile resistance at the three steps, as proposed in [21]:

$$F_{pt,1} = \sigma \left( \varepsilon_{pu,corr} \left( \frac{P_{\max\_sect,j}}{r_{out}} \right) \right) \cdot \left[ A_{res\_w} \left( \frac{P_{\max\_sect,j}}{r_{out}} \right) + 5A_{res\_w} \left( \frac{P_{av\_sect,j}}{r_{out}} \right) + A_{w,in} \right]$$
(7)a

$$F_{pt,2} = \sigma \left( \varepsilon_{pu,corr} \left( \frac{P_{av\_sect,j}}{r_{out}} \right) \right) \cdot \left[ 5A_{res\_w} \left( \frac{P_{av\_sect,j}}{r_{out}} \right) + A_{w,in} \right]$$
(7)b

$$F_{pt,3} = f_{pt} \cdot A_{w,in} \tag{7}c$$

Where  $A_{w,in}$  is the cross section of the inner uncorroded wire, while the ultimate strains of corroded wires are derived from the Eqs. (6).



Fig. 5 The *SCPS* model: (a) equivalent spring model and stress-strain relationship, (b) ultimate corroded strain of the wire,  $\varepsilon_{pu,corr}$ , vs  $P_{max\_sect,j}/r_{out}$  [16], [21].

Experimental data available in the scientific literature often define the rupture of the strand in correspondence of the failure of the first wire, that should correspond to the most corroded wire, Fig. 6(a) and Eq. (7)a. In this context, in [21], the Authors collected data on tensile tests on corroded strands and validated the *SCPS* model by comparing the analytical resistance, obtained with Eq. (7)a,  $F_{pt,l}$ , and the corresponding strain,  $\varepsilon_{pu,corr,l}$ , with the experimental values. Fig. 6(b) shows the *SCPS* model validation in terms of statistical distribution of the ratios between the analytical and experimental tensile resistance,  $F_{pt,l}$ , and ultimate strain,  $\varepsilon_{pu,corr,l}$ , evaluated in correspondence of the failure of the first wire.



Fig. 6 SCPS validation: (a) Tensile resistance – strain relation for strands, (b) statistical distribution of the ratios between the analytical and experimental tensile resistance and ultimate strain.

Note that the Eq. (7)a is valid when none of the exterior wires is characterised by 100% section loss. In the case of some wires characterised by 100% section loss at the *j*-th section, it is assumed the same pit depth for the remaining outer wires,  $P_{av\_sect,j}$ , and the inner wire uncorroded. For example, in the case of 2 wires characterised by 100% section loss at the *j*-th section, it results that the first

drop in the tensile resistance vs strain relation,  $F_{pt,l}$ , corresponds to the rupture of all the external wire and therefore corresponds to  $F_{pt,2}$ , Fig. 6(a).



Fig. 7 Relation between the tensile resistance,  $F_{ptl}$ , and the variable  $P_{max\_sect,j}/r_{out}$ .

Fig. 7 shows the relation between the tensile resistance,  $F_{ptl}$ , and the variable  $P_{max\_sect,j}/r_{out}$  at the *j*-th section. The points A, B, C and D - along the continuous green line - correspond to the tensile resistance,  $F_{ptl}$ , calculated by rearranging the Eqs. (7)a, (4), (6), (5) when  $P_{max\_sect,j}/r_{out}$  is respectively equal to 0, 0.33, 0.86, 2. Similarly, the points A', B', C' and D' - along the dotted green line - correspond, as an example, to the tensile resistance,  $F_{ptl}$ , calculated by rearranging the Eqs. (8), (4), (6) and (5) when 2 wires are characterised by 100% section loss.

# 4 Probabilistic assessment and corrosion scenarios of the bridge girder

### 4.1 Flow-chart of the probabilistic assessment procedure

The probabilistic assessment is out by adopting a Matlab code, developed for the purpose, already used in a previous study by the Authors [9] with added new features. The flow-chart of the probabilistic assessment is shown in Fig. 8.



Fig. 8 Flow-chart of the probabilistic assessment.

The Latin Hypercube Sampling (*LHS*) [8] is used as a statistical sampling technique for the random variables (both uncorroded and corroded) and 1000 analyses are run for each scenario discussed in the Paragraph 4.3.

# 4.2 Assessment of the bridge girder flexural capacity

The moment vs curvature is the output of the Matlab code used in this paper to describe the effect of the corrosion of longitudinal strands in the bottom flange of the PC bridge girder. The moment vs curvature relation is evaluated by imposing sectional equilibrium between concrete and steel rein-

forcement and by considering plane cross sections and perfect bond between steel and concrete. The random variables associated to the mechanical properties of concrete and uncorroded steel reinforcement are reported in Table 1. The non-linear behaviour of concrete in compression is described by the Mander model [23], while and a linear elastic relation till cracking, followed by a power law for softening after cracking, is used to describe the behaviour of concrete in tension [24]. The ultimate compressive strain in the concrete is equal to 0.35%. The Val model [25] and the Berrocal et al. model [26] are respectively adopted to estimate the residual area and the stress-strain relationships of corroded longitudinal bars in the bottom flange of the *PC* bridge girder. The *SCPS* model, previously described, is adopted to calculate the tensile capacity of corroded prestressing strands/wires [16].

The decay of the prestressing force is assumed proportional to the cross-section loss of the strand. It results that the prestressing force for each corroded strand can be calculated as given in Eq. (9):

$$N_{p,0,corr} = \left(A_{res\_sect,j} / A_{sect,j}\right) \cdot N_{p,0} \tag{9}$$

Where  $A_{sect,j}$  is the cross section of the uncorroded strand and  $A_{res\_sect,j}$  is the residual area of the corroded strand calculated as given in Eq. (10):

$$A_{res\_sect,j} = A_{res\_w} \left( \frac{P_{\max\_sect,j}}{r_{out}} \right) + 5 \cdot A_{res\_w} \left( \frac{P_{av\_sect,j}}{r_{out}} \right) + A_{w,in}$$
(10)

The term  $N_{p,0} = \sigma_{p0} \cdot A_{sect,i}$  is the initial prestress force applied to each strand.

If the prestress strain,  $\varepsilon_{p0}$ , is greater than the ultimate strain of a corroded wire,  $\varepsilon_{pu,corr}$ , this wire prematurely fails and must be removed from the calculation of the flexural capacity. This condition can be derived, by imposing in Eq. (6)b the condition  $\varepsilon_{pu,corr}=\varepsilon_{p0}$  in the range  $P_{max\_sect,j}/r_{out} > 0.33$ . It results that for pits dept of the wire higher than the value given Eq. (11), a premature and brittle rupture of the wire occurs only due to the applied prestressing force:

$$\left[\frac{P_{\max w,j}}{r_{out}}\right]_{p0} \ge \left[0.33 + \frac{1}{0.599} \left(1 - \frac{\varepsilon_{p0}}{\varepsilon_{py}}\right)\right] \tag{11}$$

It is important to point out that, as the applied prestressing force increases - and consequently as the value of  $\varepsilon_{p0}$  increases -, as the corrosion level, defined in terms of  $P_{max\_w,j}$ , that cause premature and brittle wire's rupture decreases. Therefore, the fragility of *PC* members is not only related to the corrosion level but also to the value of the prestressing strain applied in strands.

### 4.3 Corrosion scenarios: position, extension, intensity of the corrosion damage

In this work, corrosion scenarios are assumed and applied to the selected case study without any reference to the actual situation of the viaduct, which is unknown to Authors. Therefore, the selected case study represents only an example for the application of a general procedure that Authors are presenting for prestressed bridge girders. The corrosion scenarios are described based on beam's survey during visual inspections when no data about measurement of the corrosion in prestressing strands are available.

# 4.3.1 Position of the corrosion damage

Two different positions of the corrosion damage in longitudinal prestressing strands placed at the bottom flange of the bridge girder are analysed, as illustrated in Fig. 2: section A - A (having distance equal to L/8 from the support) and section B - B (at midspan).

### 4.3.2 Extension of the corrosion damage

As illustrated in Fig. 9, the prestressing reinforcement of bridge girders is usually characterised by several layers of strands. Since in this paper the capacity of the analysed prestressed beam is correlated to the damage detected via visual inspection, no assumptions about the corrosion of internal layers are made. Therefore, internal strands are considered uncorroded.

The extension levels of corrosion in external strands, are defined by assuming that a critical chloride content is reached in the concrete part highlighted in yellow in Fig. 9. Certainly, additional data, for example based on the measurement of the chloride content, could allow to plot the chloride content profile and establish if concrete around internal layers of strands could had reached a critical value.

Consequently, the extension levels of corrosion are established based on the ratio between the corroded and "visible" strands to the total number of external strands. For example, at Section B-B where 16 external strands can be identified, Fig. 9 shows that extension levels of corrosion equal to (i) 25 %, (ii) 50 %, (iii) 100 % correspond to 4, 8, 16 corroded "visible" strands.



Fig. 9 Definition of the extension level,  $k_{ext}$ , of the corrosion in strands at the bottom flange according (example at Section B-B).

### 4.3.3 Intensity of the corrosion damage

According to Tuutti model [27], Fig. 10, shows the approach proposed for the definition of the intensity level of the corrosion in strands at the bottom flange. Three different levels of intensity are assumed in correspondence of three different damages that can be qualitatively noted during visual inspections:

- (1) visible splitting cracks,
- (2) concrete cover spalling and visible section loss in corroded wires,
- (3) 100% cross-section loss of some wires in strands.

Certainly, there is not unique match between these latter qualitative observations and the quantitative values of the pit depth in wires of strands. In this paper the following damage indicators are considered representative examples of the three damage levels qualitatively observed during in-situ inspections:

- (1) visible splitting cracks: crack opening width, *w*, equal to 0.3 mm,
- (2) concrete cover spalling and visible section loss in corroded wires: crack opening width, w, equal to 1.5 mm (according to [28], cover spalling is assumed to occur for crack opening width higher than 1 mm),
- (3) 100% cross-section loss of some wires in strands: 2 wires having 100% section loss.

In future, further studies will be carried out by Authors to optimise the boundary conditions of the adopted damage indicators.

Since the paper aims to find a correlation between the proposed levels of corrosion intensity/extension and the flexural capacity of the *PC* bridge girder, in the following, the damage indicators adopted to represents the three different intensity levels are related to mean values of the variable  $P_{\text{max}\_sect,j}/r_{out}$ . In particular, the probability distribution functions shown in Fig. 3(e), related to mean values,  $\mu(P_{max\_sect,j})/r_{out}$ , equal to 0.14, 0.43 and 0.82 are used to describe the three different intensity levels of corrosion in longitudinal strands placed at the bottom flange of a bridge girder. In the following, the procedure adopted to define the value of  $\mu(P_{max\_sect,j})/r_{out}$  corresponding to the low  $(\mu(P_{max\_sect,j})/r_{out} = 0.14)$ , medium  $(\mu(P_{max\_sect,j})/r_{out} = 0.43)$ , high  $(\mu(P_{max\_sect,j})/r_{out} = 0.82)$  intensity levels is described.

For low and medium intensity levels, the relation between corrosion in strands and crack opening width proposed by Ahmed et al. [29] has been modified to be directly implemented in *SCPS* model. The relationship proposed by Ahmed et al. [29] depends on the prestress  $\sigma_{p0}$ , the water-cement ratio  $R_{w/c}$ , and the cover-diameter ratio,  $c/\phi_p$ , as given in the Eq. (12):

$$w = 0.16 \frac{e^{\binom{\sigma_{p_0}}{f_{p_t}}}}{R_{w/c}} \left(P_{sum\_sect,j} - P_0\right)$$
(12)

Being  $P_0$  the sum of the pit depths in external wires corresponding to splitting crack initiation, calculated as given in Eq. (13):

$$P_0 = 0.036 \frac{R_{w/c}}{e^{\binom{\sigma_{p_0}}{f_{p_t}}}} \left[\frac{c}{\phi_p}\right]^2 + 0.128$$
(13)

In this paper,  $R_{wc}$  is equal to 0.5, the cover is equal to 50 mm ( $c/\phi_p = 3.3$ ), and the prestress ratio,  $\sigma_{p0}/f_{pt}$ , is equal to 966.18/1926 = 0.50. The sum of the pit depth in external wires,  $P_{sum\_sect,j}$ , is obtained, according to the *SCPS* model, as given in Eq. (14):

$$P_{sum\_sect,j} = \mu (P_{max\_sect,j}) + 5 \cdot [P_{av\_sect,j}]_{\mu (P_{max\_sect,j})}$$
(14)

Where  $\mu(P_{\max\_sect,j})$  is calculated by multiplying the average ratio  $\mu(P_{\max\_sect,j})/r_{out}$  time the outer wire radius (in this case,  $r_{out}$  is equal to 0.5.5.01 = 2.505 mm), while the average pit depth of the remaining exterior wires is calculated as given in Eq. (15):

$$\left[P_{av\_sect,j}\right]_{\mu(P_{\max\_sect,j})} = r_{out} \left[0.378 \left(\frac{\mu(P_{\max\_sect,j})}{r_{out}}\right)^2 + 0.25 \frac{\mu(P_{\max\_sect,j})}{r_{out}}\right]$$
(15)



Fig. 10 Definition of the intensity levels: (a) first approach,  $k_{int, I}$ , and (b) second approach,  $k_{int, 2}$ , of the corrosion in strands at the bottom flange.

A mean value of the variable  $\mu(P_{max\_sect,j})/r_{out} = 0.14$  for low intensity level is obtained by inputting in Eqs. (12)-(15) the assumed crack opening width value w = 0.3 mm. A mean value of the variable  $\mu(P_{max\_sect,j})/r_{out} = 0.43$  for medium intensity level is obtained by inputting in Eqs. (12)-(15) the assumed crack opening width value w = 1.5 mm. Since 100% section loss of 2 wires characterises the damage indicators of the high intensity level, an average value of the pit depths is assigned to the four external remaining wires by using the Eq. (15) with  $\mu(P_{max\_sect,j})/r_{out} = 0.82$ , while the inner wire is kept uncorroded [19]. At the University of Parma, the analysis of 3D-scans carried out on strands characterised by wires having 100% section loss are ongoing now; definitely, the analyses of these data will help in selecting the appropriate value  $\mu(P_{max\_sect,j})/r_{out}$  that in this study is selected quite arbitrarily.

Two approaches are proposed to assign a value to the intensity level,  $k_{int}$ , of the corrosion scenarios.

The first approach assigns a value to the intensity level,  $k_{int,I}$ , only based on the qualitative observation of the damage as follow:

a. visible splitting cracks:  $k_{int,1} = 0.25$ ,

(i)

- b. concrete cover spalling and visible section loss in corroded wires:  $k_{int,l} = 0.5$ ,
- c. 100% cross-section loss of some wires in strands:  $k_{int,1} = 1$

(ii) The second approach assigns a value to the intensity level,  $k_{int,2}$ , based on the ratio between the tensile resistance of the corroded strand,  $F_{ptl}$ , (as defined in Paragraph 3.2) and the tensile resistance of the uncorroded strand,  $F_{pt0}$ , as given in Eq. (16):

$$k_{int,2} = 1 - \frac{F_{pt,1}}{F_{pt,0}} \tag{16}$$

The assigned values of the intensity level result as follows:

- a. visible splitting cracks:  $k_{int,2} = 0.06$
- b. concrete cover spalling and visible section loss in corroded wires:  $k_{int,2} = 0.20$
- c. 100% cross-section loss of some wires in strands:  $k_{int,2} = 0.56$

In Paragraph 6, the value of the intensity level,  $k_{int,2}$ , is adopted to propose a suitable defect-based index for the simplified calculation of the resistant moment of *PC* bridge girders.

### 4.3.4 Corrosion scenarios

The list of the corrosion scenarios considered for the external strands at bottom flange is reported in Table 2. The corrosion is considered also in longitudinal rebars at bottom flange, by assuming the same pit depth of the prestressing steel. Since the contribute of longitudinal bars on the overall resistant moment is low, this arbitrary assumption doesn't strongly affect the obtained results.

Table 2 Corrosion scenarios for Sections A-A and B-B and pit depth statistical distribution for the different intensity levels.

Low and medium intensity levels							
k <sub>int, 1</sub>	k <sub>int,2</sub>	Distribution					
0.25	0.06	0.25	0.14	0.07			
0.50	0.20	Logn					
1.00	0.56	1.00	0.82	0.25			
*Mean	*Mean and standard deviations used as statistical input for pit depth, $r_{out} = 2.505$ mm						

As input in the Matlab program, in addition to uncorroded properties, the distribution parameters reported in Table 2 are adopted. A total of 20 scenarios are considered: 9 (3  $k_{ext}$  times 3  $k_{int}$ ) + 1 uncorroded scenario, times 2 sections (A-A and B-B).

# 5 Results

# 5.1 Flexural capacity: moment vs curvature relations

Fig. 11 shows the decay of the flexural capacity (in terms of moment vs curvature relation) calculated at Section B-B for the highest levels of extension,  $k_{ext} = 1$  and variable intensity levels  $k_{int,1} = 0.25, 0.5, 1.00$  with the first approach. For a given extension level, the comparison between the moment vs curvature response of the uncorroded, Fig. 11(a), and the corroded, Fig. 11(b)-(e), bridge girder shows:

- 1) A loss of resistant moment (approximately till 30%), Fig. 11(a) (d). The peak value of the resistant moment for uncorroded and corroded bridge girder corresponds to the achievement of the ultimate strain in external strands that are the farthest from the neutral axis.
- 2) A decay in the ductility and curvature at the maximum bending moment (approximately till 50%). The loss in curvature is calculated as  $\Delta \chi_{max} = \chi_{u,0} \chi_{u,corr}$ , being  $\chi_{u,0}$  and  $\chi_{u,corr}$  the curvature, predicted in correspondence of the peck value of the moment resistance, respectively, for the uncorroded and corroded bridge girder, Fig. 11(g);
- 3) A decay in the mean value of the resistant moment proportional to the corrosion intensity level; while the standard deviation,  $\sigma_M$ , of the probability distribution of the resistant moment, increases with respect to the uncorroded beam but not proportionally to the corrosion intensity level Fig. 11(e)

Similar results are shown in Fig. 12 for Section A-A and  $k_{ext} = 1$ ,  $k_{int,l} = 1$ .



Fig. 11 Moment vs curvature relations at Section B-B for (a) uncorroded strands, (b)-(d) corroded strands ( $k_{ext}$ =1,  $k_{int,1}$  = 0.25, 0.5, 1.00), (e) resistant moment probability distribution, (f) cross section and strands' layout, (g) loss in curvature measured at peak of the moment resistance.



Fig. 12 Moment vs curvature relations at Section A-A for (a) uncorroded and (b) corroded  $(k_{exr}=1, k_{int,l}=1)$  bridge girders, (c) Cross section and strands' layout.

The decay in the resistant moment at sections B-B and A-A is shown in Fig. 13 for all the analysed corrosion scenarios. The loss of resistant moment  $\Delta M_{max}$  is normalised with respect to the maximum resistant moment of the uncorroded bridge girder,  $M_{max,0}$ . It can be observed in Fig. 13(b) that for the same levels of corrosion extension and intensity, in strands placed at the bottom flange of the cross section, the loss of resistant moment is a bit lower for section B-B (44 strands are placed in the bottom flange) than in section A -A (26 strands are placed in the bottom flange).



Fig. 13 (a) Layers of strand, decay of maximum resistant moment due to corrosion of strands at bottom flange at (b) Section B-B and (c) Section A-A.

Table 3 reports the mean values and the standard deviation values of the probability distribution of the resistant moment at Section B-B and Section A-A resulting from Montecarlo analysis. Table 3 shows that, given the same level of corrosion extension, the standard deviation is lower for the highest intensity level,  $k_{int,I} = 1$ , than for the low,  $k_{int,I} = 0.25$ , and medium,  $k_{int,I} = 0.5$ , intensity level.

			Sectio	n B-B	Section A-A	
k <sub>ext</sub>	k <sub>int, 1</sub>	k <sub>int,2</sub>	$\mu_M$ [kN·m]	σ <sub>M</sub> [kN·m]	$\mu_M [\mathrm{kN} \cdot \mathrm{m}]$	$\sigma_M [kN \cdot m]$
0	0	0.00	30631.40	176.78	22326.18	118.89
	0.25	0.06	30098.64	185.49	21954.68	132.61
0.25	0.50	0.20	28722.55	197.80	20967.87	146.64
	1.00	0.56	28528.80	173.91	20744.67	110.31
	0.25	0.06	29748.57	201.00	21721.68	143.04
0.50	0.50	0.20	28200.61	263.01	20555.17	199.02
	1.00	0.56	26419.79	176.50	19152.63	107.42
1.00	0.25	0.06	29276.92	225.73	21421.85	155.92
	0.50	0.20	26434.76	283.51	19438.34	250.66
	1.00	0.56	22230.56	208.19	16006.96	115.31

Table 3 Data of the resistant moment distribution at Section B-B and Section A-A.

The lowest dispersion of resistant moment for high intensity level of corrosion, can be attributed to the lower number of corroded wires contributing to the flexural capacity. Indeed, 2 wires are considered missing as damage indicator of high intensity level,  $k_{int,I} = 1$ . Furthermore, the mean value,  $\mu(P_{max\_sect,j})/r_{out} = 0.82$ , applied for high intensity level to the remaining external wires corresponds to a frequent occurrence in the Montecarlo analysis of wires' removal because not able to sustain the prestressing force, as explained in Paragraph 4.2.

For all the scenarios, the maximum resistant moment is achieved both for uncorroded and corroded bridge girder at the attainment of the ultimate tensile strain values in external strands. In previous works, [9], published by Authors, and aimed to analyse the flexural capacity of naturally corroded prestressed beams characterised by a rectangular cross section and reinforced only with two prestressing strands, corrosion caused a change in the failure mode (due to concrete crushing for uncorroded beams and due to wire rupture in corroded beams) and a higher difference in the dispersion of the resistant moment values for uncorroded and corroded beams. This latter remark highlights that the effect of the corrosion of strands on moment resistance, ductility and related uncertainties strongly depends on the layout of the prestressing reinforcement (i.e. number of strands, position of layers along the depth of the beam, etc.) and on the shape and dimension of the cross section.

### 5.2 Design value of resistant moment

The design value of resistant moment,  $M_{Rd}$ , can be estimated by intersecting the related cumulative distribution functions (*CDF*) with percentiles calculated as:

$$p = \Phi(-\alpha\beta) \tag{17}$$

where  $\Phi$  is the cumulative standard normal distribution and  $\alpha$  is the sensitivity factor [15].



Fig. 14 (a) Estimation of the design value of resistant moment, (b) *PDF* and *CDF* of resistant moment at Section B-B ( $k_{ext} = 1$  and  $k_{int,I} = 1$ ).

An illustrative example is reported in Fig. 14(a).

Table 4 Design value of resistant moment and applied moment at Sections B-B and A-A.

			Section B-B	Section A-A
k <sub>ext</sub>	$k_{int, l}$	$k_{int,2}$	$M_{Rd}$	$M_{Rd}$
			[kN·m]	[kN·m]
0	0	0.00	30216.02	22061.85
	0.25	0.06	29641.36	21640.19
0.25	0.50	0.20	28268.47	20628.32
	1.00	0.56	28128.39	20478.79
	0.25	0.06	29262.77	21377.22
0.50	0.50	0.20	26962.54	19771.43
	1.00	0.56	25980.56	18886.33
	0.25	0.06	28748.73	21022.50
1.00	0.50	0.20	25764.14	18738.38
	1.00	0.56	21804.18	15774.42

A large relative cost of safety measures and a consequence class 3 is considered appropriate for the bridge case study, consequently the target reliability index is equal to 3.7 [15], [30]. For a reference period of 1 year, the sensitivity factors for resistance,  $\alpha_R$ , is equal to 0.7, leading to resistance and demand percentiles equal to 0.48%. In the previous work presented by Authors [9], the Generalised Extreme Value (*GEV*) distribution results appropriate to fit the resistance of corroded members, since extreme values of the distribution can be detected (as shown in Fig. 14(b)). Table 4 reports the design value of resistant moment vs extension,  $k_{ext}$ , and intensity,  $k_{int}$  parameters.

Fig. 15 shows the dependency of the decay of the design value of the resistant moment on the intensity and extension levels of corrosion observed during visual in-situ inspections, according to the proposed procedure and the damage indicators selected to develop the presented case study.



Fig. 15 Decay of the design value of the resistant moment depending on the intensity and extension levels of corrosion observed during visual in-situ inspections.

Certainly, by selecting different values of damage indicators, to define the intensity level of corrosion (such as different crack opening width values associated to splitting cracks of cover spalling, for low and medium level, or different numbers of missing wires for high level) the observed decay in design value of resistant moment could be different. Anyway, the proposed procedure for scaling the extension and intensity levels, in function of the visible damage induced by corrosion - as detected in Level 1 - (i.e. splitting crack, cover spalling and evident section loss in wire, wire rupture), provides consistent results in terms of gradual reduction of resistant moment.

# 6 Suitable defect-based index for the calculation of the resistant moment

A defect-based index is proposed for a simplified calculation of the resistant moment of *PC* bridge girders subjected to corrosion of longitudinal strands in the bottom flange and characterised by flexural failure due to strands' rupture.

The resistant moment of the bridge girder is the sum of the resistant moment provided by internal prestressing strands (assumed unaffected by corrosion),  $\sum_{i=1}^{N-N_{ext}} M_{Rp,i}$ , external prestressing strands (assumed potentially affected by corrosion),  $\sum_{i=1}^{N_{ext}} M_{Rp,i}$ , and bars,  $\sum_{i=1}^{N_b} M_{Rb,i}$ , Eq. (18):

$$M_{R} = \sum_{i=1}^{N-N_{ext}} M_{Rp,i} + \sum_{i=1}^{N_{ext}} M_{Rp,i} + \sum_{i=1}^{N_{b}} M_{Rb,i}$$
(18)

Being N the total number of prestressing strands,  $N_{ext}$  the total number of external prestressing strands (as defined in Paragraph 4.3.2) and  $N_b$  the total number of bars.

To simplify the calculation, only external prestressing strands are considered affected by corrosion while external bars are considered uncorroded. Hence, the difference between the resistant moment of the corroded and uncorroded bridge girder is due to the resistant moment provided by external prestressing strands as given in Eq. (19):

$$\sum_{i=1}^{N_{ext}} M_{Rp,i} = \left(1 - k_{int,2} \cdot k_{ext}\right) \sum_{i=1}^{N_{ext}} \left(F_{pt,0} \cdot z_i\right)$$
(19)

where  $z_i$  is the internal lever arm of prestressing strands, calculated with respect to the position of the resultant of the compressive stresses in concrete. Since the neutral axis is located at a distance equal to  $0.5 \cdot t_{slab}$  from the beam's extrados, it results that - for the geometry of the analysed section -  $z_i = d_i - 0.25 \cdot t_{slab}$ , where  $d_i$  is the distance of *i*-th strand from the beam's extrados e and  $t_{slab}$  is the slab's thickness. Eq. (19) is valid both for uncorroded ( $k_{int,2} = 0$ ) and corroded scenarios.

Table 5 provides the resistant moment,  $M_{R,Eq,(18)}$ , - at Sections B-B and A-A - calculated by substituting Eq. (19) in Eq. (18) and by adopting the mean value of the mechanical properties of materials, Table 1. The ratios between the resistant moment calculated with Montecarlo analysis,  $\mu_M$ , (reported in Table 3) and the resistant moment calculated with Eqs. (18)-(19),  $M_{R,Eq,(18)}$ , are listed in Table 5.

Table 5 shows that the proposed simplified procedure seems a suitable tool for a quick estimation of the flexural capacity of corroded prestressed bridge girder, although an accurate measurement of corrosion-induced damage and a refined level of analysis is always required.

		$1 - k_{int,2} \cdot k_{ext}$	Sect	ion B-B	Section A-A	
k <sub>ext</sub>	k <sub>int,2</sub>		$[kN \cdot m] M_{R,Eq.(18)}$	$\mu_M / M_{R, Eq.(18)}$	$\frac{M_{R,Eq.(18)}}{[\text{kN}\cdot\text{m}]}$	$\mu_M / M_{R, Eq.(18)}$
0.00	0.00	1.00	30082.54	1.02	21485.92	1.04
	0.06	0.98	29920.43	1.01	21323.82	1.03
0.25	0.20	0.95	29559.02	0.97	20962.41	1.00
	0.56	0.86	28644.32	1.00	20047.70	1.03
0.50	0.06	0.97	29758.33	1.00	21161.72	1.03
	0.20	0.90	29035.50	0.97	20438.89	1.01
	0.56	0.72	27206.10	0.97	18609.48	1.03
1.00	0.06	0.94	29434.12	0.99	20837.51	1.03
	0.20	0.80	27988.47	0.94	19391.86	1.00
	0.56	0.44	24329.65	0.91	15733.04	1.02
			Mean:	0.979	Mean:	1.021
			St. Dev.:	0.031	St. Dev.:	0.014

Table 5 Mean values of the resistant moment at Sections B-B and A-A.

In Table 5 the design values of the resistant moment are not reported because the geometrical uncertainties, the model uncertainties and the uncertainties related to the attained level of knowledge were not considered. The model uncertainties were considered in previous studies to calibrate partial safety factors for corroded strands, [31], but for a more comprehensive evaluation of all the involved uncertainties, additional studies are required.

### Conclusions

In this work, the flexural capacity of prestressed I-shaped concrete bridge girders subjected to corrosion in longitudinal strands placed at the bottom flange is analysed. In the following the main outcomes are listed:

- A procedure for the qualitative assignment of the corrosion level in terms of extension and intensity to be adopted during visual inspection is proposed,
- Damage indicators are selected, as an example, to quantitatively evaluate the flexural capacity corresponding to the selected levels of extension and intensity of the corrosion,
- A robust probabilistic approach is carried out by applying the *SCPS* model for the evaluation of the tensile capacity of corroded strands,
- The *PDF* of the resistant moment obtained with Montecarlo analysis allows to evaluate the design value of the resistant moment, although uncertainties related to the bridge geometry and the attainment of the level of knowledge are not considered in this paper,
- A defect-based index is proposed for a simplified calculation of the resistant moment of *PC* bridge girders subjected to corrosion of longitudinal strands in the bottom flange and characterised by flexural failure due to strands' rupture,
- The simplified procedure seems a suitable tool for a quick estimation of the flexural capacity of corroded prestressed bridge girder.

At the University of Parma further research is ongoing to improve the adopted procedure and to extend it to the analysis of different defects related to corrosion of strands, such as corrosion in deviated tendons, corrosion in post-tensioned wires, etc.

# Acknowledgements

The presented study is part of a program of activities funded by the Italian "Consiglio Superiore dei Lavori Pubblici". The results were achieved in the national technical agreement for implementing the agreement pursuant to art. 15 law 7 August 1990, No. 241 between the Superior Council of Public Works and RELUIS.

# References

- A. Nettis, A. Nettis, S. Ruggieri, and G. Uva, "Corrosion-induced fragility of existing prestressed concrete girder bridges under traffic loads," Eng Struct, vol. 314, no. May, p. 118302, 2024.
- [2] P. P. Rossi, N. Spinella, and A. Recupero, "Experimental application of the Italian guidelines for the risk classification and management and for the safety evaluation of existing bridges," Structures, vol. 58, no. July, p. 105387, 2023.
- [3] MIT, "DM. 01 July 2022. Guidelines for the classification and risk Management, safety assessment and monitoring of existing bridges," Italian High Council of Public Works (in italian), 2022.
- [4] P. Saura Gomez et al., "Experimental correlations between crack opening and corrosion measurements in prestressed concrete beams," in SMAR 2024 – 7th International Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures, 4-6 September 2024, Salerno, Italy, 2024.
- [5] B. Belletti, J. Rodríguez, C. Andrade, L. Franceschini, J. Sánchez Montero, and F. Vecchi, "Experimental tests on shear capacity of naturally corroded prestressed beams," Structural Concrete, vol. 21, no. 5, pp. 1777–1793, 2020.
- [6] L. Wang, T. Li, L. Dai, W. Chen, and K. Huang, "Corrosion morphology and mechanical behavior of corroded prestressing strands," Journal of Advanced Concrete Technology, vol. 18, no. 10, pp. 545–557, 2020.
- [7] F. Vecchi, L. Franceschini, F. Tondolo, B. Belletti, J. Sánchez Montero, and P. Minetola, "Corrosion morphology of prestressing steel strands in naturally corroded PC beams," Constr Build Mater, vol. 296, p. 123720, 2021.
- [8] A. Olsson, G. Sandberg, and O. Dahlblom, "On Latin hypercube sampling for structural reliability analysis," Structural Safety, vol. 25, no. 1, pp. 47–68, 2003.
- [9] B. Belletti, R. Caspeele, W. Botte, L. Franceschini, S. Ravasini, and S. Sandrini, "Probabilistic assessment of the moment-curvature response of PC beams with different corrosion scenarios," in IABMAS Conference 2024, June 24th – June 28th 2024, Copenhagen, Denmark, 2024, pp. 965–973.

- [10] P. C. Gallo and M. P. Petrangeli, "The viaduct crossing of the Calabria divide between Gioiosa Jonica and Rosarno," Industria Italiana del Cemento, vol. 53, pp. 309–332, 1983.
- [11] JCSS, "Probabilistic Model Code," 2001, Joint Committee on Structural Safety.
- [12] G. Miluccio, D. Losanno, F. Parisi, and E. Cosenza, "Traffic-load fragility models for prestressed concrete girder decks of existing Italian highway bridges," Eng Struct, vol. 249, no. September, p. 113367, 2021.
- [13] G. M. Verderame, P. Ricci, M. Esposito, and F. C. Sansiviero, "Le caratteristiche meccaniche degli acciai impiegati nelle strutture in C.A. realizzate dal 1950 al 1980," in XXVI Convegno Nazionale AICAP "Le prospettive di sviluppo delle opere in calcestruzzo strutturale nel terzo millennio," 2011, pp. 1–8.
- [14] L. Jacinto, M. Pipa, L. A. C. Neves, and L. O. Santos, "Probabilistic models for mechanical properties of prestressing strands," Constr Build Mater, vol. 36, pp. 84–89, 2012.
- [15] R. Caspeele and K. Van Den Hende, "Validation of the harmonized partial factor method for design and assessment of concrete structures as proposed for fib model code 2020," Structural Concrete, vol. 24, no. 4, pp. 4368–4376, 2023.
- [16] L. Franceschini, B. Belletti, F. Tondolo, and J. Sanchez Montero, "A simplified stress-strain relationship for the mechanical behaviour of corroded prestressing strands: the SCPSmodel," Structural Concrete, vol. 24, no. 1, pp. 189–210, 2022.
- [17] H. Zhou, S. Wan, and W. Li, "Test and simulation of corroded high strength steel wires: From scanned morphology feature to mechanical degradation," Corros Sci, vol. 240, no. August, p. 112392, 2024.
- [18] "fib 'Corroded prestressing database."" [Online]. Available: https://www.fibinternational.org/commissions/databases.html
- [19] A. Sirico, O. Palii, S. Ravasini, B. Belletti, and J. Sanchez Montero, "Pitting morphology of prestressing strands subjected to naturally chloride environment," in submitted to Proceedings of the fibCACRCS 2025, June 30 – July 3, 2025, Lecco, Italy, 2025.
- [20] L. Franceschini, B. Belletti, and F. Tondolo, "Study on the Probability Distribution of Pitting for Naturally Corroded Prestressing Strands Accounting for Surface Defects," Buildings, vol. 12, no. 1732, pp. 1–19, 2022.
- [21] S. Ravasini and B. Belletti, "An overview of constitutive models for corroded strands," in Italian Concrete Conference, Florence, 19-21 June 2024, 2024, pp. 141–148.
- [22] L. Franceschini, F. Vecchi, F. Tondolo, B. Belletti, and J. Sánchez Montero, "Mechanical behaviour of corroded strands under chloride attack: A new constitutive law," Constr Build Mater, vol. 316, 2022.
- [23] J. B. Mander, M. J. N. Priestley, and R. Park, "Theoretical Stress-Strain Model for Confined Concrete," Journal of Structural Engineering, vol. 114, no. 8, pp. 1804–1826, 1988.
- [24] J. Dong Lee, "The effect of tension stiffening in moment-curvature responses of prestressed concrete members," Eng Struct, vol. 257, no. January, p. 114043, 2022.
- [25] D. V. Val, "Deterioration of Strength of RC Beams due to Corrosion and Its Influence on Beam Reliability," Journal of Structural Engineering, vol. 133, no. 9, pp. 1297–1306, 2007.
- [26] C. G. Berrocal, E. Chen, I. Löfgren, and K. Lundgren, "Analysis of the flexural response of hybrid reinforced concrete beams with localized reinforcement corrosion," Structural Concrete, no. December 2022, pp. 1–24, 2023.
- [27] K. Tuutti, "Corrosion of steel in concrete," PhD Thesis, Swedish Cement and Concrete Research Institute, Stockholm, 1982.
- [28] "DuraCrete Final Technical Report," 2000.
- [29] M. Ahmed, A. K. El-Sayed, A. M. Alhozaimy, A. I. Al-Negheimish, and A. Alabdulkarim, "Prediction of strand corrosion damage in prestressed concrete beams through visual inspection of corrosion concrete cracking," Constr Build Mater, vol. 416, no. February, p. 135232, 2024.
- [30] ISO 2394, "General Principles of Reliability for Structures," 2015, Geneva, Switzerland.
- [31] L. Franceschini, R. Caspeele, B. Belletti, F. Tondolo, and J. Sanchez, "Partial Safety Factor for the Design Strength Prediction of Naturally Corroded Prestressing Strands," in 14th International Conference on Applications of Statistics and Probability in Civil Engineering, ICASP14 Dublin, Ireland, July 9-13, 2023, 2023, pp. 1–8.