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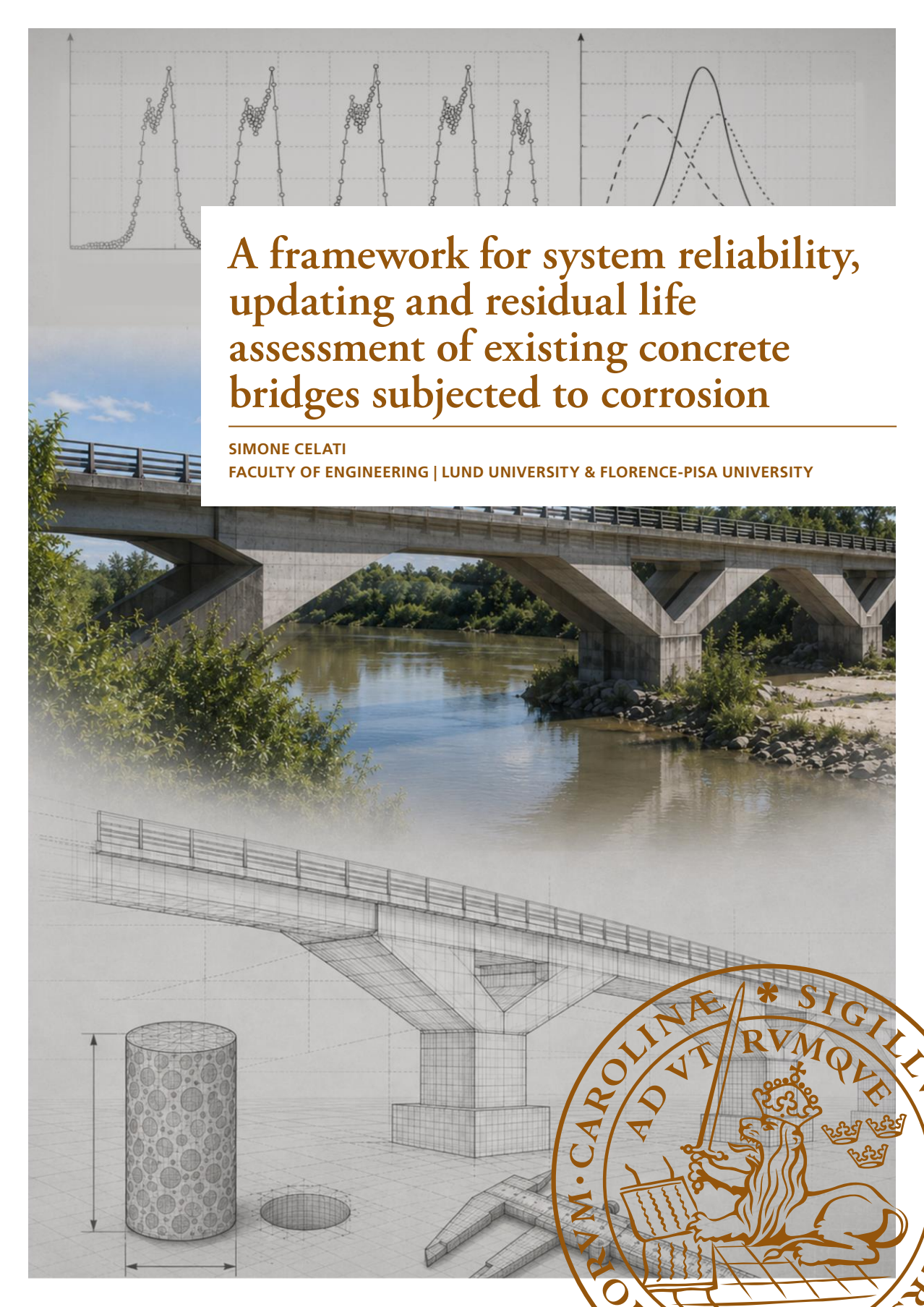
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A framework for system reliability, updating and residual life assessment of existing concrete bridges subjected to corrosion

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A framework for system reliability, updating and residual life assessment of existing concrete bridges subjected to corrosion

Simone Celati



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Abstract: The growing age of bridge inventories worldwide demands a systematic evaluation of their safety and estimation of their remaining service life. At the same time, the pursuit of sustainability and cost efficiency compels infrastructure managers to prolong the use of existing structures, thereby reducing greenhouse gas emissions and optimising financial resources. To achieve these objectives, assessment procedures must quantify uncertainty and guide rational strategies for inspection, testing, monitoring, intervention planning and design.

This thesis presents a comprehensive system reliability and updating framework for the service-life assessment of concrete bridges affected by corrosion. The framework integrates probabilistic modelling with inspection, testing, and monitoring data, enabling the formulation of models that reflect actual degradation.

The proposed framework advances existing approaches by modelling bridges as nested systems of failure modes, sections, and elements, capturing dependencies and interactions among degradation mechanisms. Using survival information and heterogeneous data sources enables probabilistic estimates of residual life that can meaningfully inform maintenance prioritisation and planning.

The applicability and generality of the framework are demonstrated through two case studies on a post-tensioned and a reinforced concrete deck. For post-tensioned concrete bridges, it captures the occurrence of defects and spatially variable corrosion, demonstrating how probabilistic modelling can refine service-life estimates and optimise maintenance planning. For reinforced concrete bridges, the framework incorporates monitoring data through Bayesian updating, reducing uncertainty and revealing the potential for service-life extension that deterministic or prior assessments could not demonstrate.

In both cases, sensitivity and spatial analyses highlight that inspection strategies should adapt to the extent of degradation: initially focusing on the most utilised sections while, as degradation spreads, progressively involving the entire girder. These results support adaptive inspection planning that focuses first on highly stressed sections and gradually extends to other regions as damage propagates.

Integrating the reliability calculations provided and the sensitivity analysis outcomes can provide methodological, managerial, and societal contributions by promoting data-driven, cost-effective, and sustainable bridge management while maintaining safety consistent with international reliability standards.

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International Doctoral Course in Civil and Environmental Engineering

A framework for system reliability, updating and residual life assessment of existing concrete bridges subjected to corrosion

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Pisa, October 2026

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Simone Celati



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Abstract

The growing age of bridge inventories worldwide demands a systematic evaluation of their safety and estimation of their remaining service life. At the same time, the pursuit of sustainability and cost efficiency compels infrastructure managers to prolong the use of existing structures, thereby reducing greenhouse gas emissions and optimising financial resources. To achieve these objectives, assessment procedures must quantify uncertainty and guide rational strategies for inspection, testing, monitoring, intervention planning and design.

This thesis presents a comprehensive system reliability and updating framework for the service-life assessment of concrete bridges affected by corrosion. The framework integrates probabilistic modelling with inspection, testing, and monitoring data, enabling the formulation of models that reflect actual degradation.

The proposed framework advances existing approaches by modelling bridges as nested systems of failure modes, sections, and elements, capturing dependencies and interactions among degradation mechanisms. Using survival information and heterogeneous data sources enables probabilistic estimates of residual life that can meaningfully inform maintenance prioritisation and planning.

The applicability and generality of the framework are demonstrated through two case studies on a post-tensioned and a reinforced concrete deck. For post-tensioned concrete bridges, it captures the occurrence of defects and spatially variable corrosion, demonstrating how probabilistic modelling can refine service-life estimates and optimise maintenance planning. For reinforced concrete bridges, the framework incorporates monitoring data through Bayesian updating, reducing uncertainty and revealing the potential for service-life extension that deterministic or prior assessments could not demonstrate.

In both cases, sensitivity and spatial analyses highlight that inspection strategies should adapt to the extent of degradation: initially focusing on the most utilised sections while, as degradation spreads, progressively involving the entire girder. These results support adaptive inspection planning that focuses first on highly stressed sections and gradually extends to other regions as damage propagates.

Integrating the reliability calculations provided and the sensitivity analysis outcomes can provide methodological, managerial, and societal contributions by promoting data-driven, cost-effective, and sustainable bridge management while maintaining safety consistent with international reliability standards.

Sammanfattning - Swedish Summary

Den ökande åldern hos brobestånden världen över kräver en systematisk utvärdering av deras säkerhet och en uppskattning av deras återstående livslängd. Samtidigt medför strävan efter hållbarhet och kostnadseffektivitet att infrastrukturförvaltare behöver förlänga användningen av befintliga konstruktioner och därigenom minska utsläppen av växthusgaser samt optimera användningen av ekonomiska resurser. För att uppnå dessa mål måste bedömningsförfaranden kvantifiera osäkerheter och ge stöd för rationella strategier avseende inspektion, provning, övervakning, planering av åtgärder och dimensionering.

Denna avhandling presenterar ett övergripande ramverk för systemtillförlitlighet och probabilistisk uppdatering vid livslängdsbedömning av betongbroar som påverkas av korrosion. Ramverket integrerar probabilistisk modellering med data från inspektioner, provningar och övervakning, vilket möjliggör formulering av modeller som återspeglar den faktiska nedbrytningen.

Det föreslagna ramverket vidareutvecklar befintliga metoder genom att modellera broar som nästlade system av brottmoder, tvärsnitt och konstruktionselement, samtidigt som beroenden och samverkan mellan olika nedbrytningsmekanismer beaktas. Genom att använda överlevnadsinformation och heterogena datakällor kan probabilistiska uppskattningar av den återstående livslängden erhållas, vilka på ett meningsfullt sätt kan ligga till grund för prioritering och planering av underhållsåtgärder.

Ramverkets tillämpbarhet och generalitet demonstreras genom två fallstudier av ett efterspänt respektive ett armerat betongbrodäck. För efterspända betongbroar beaktar ramverket förekomsten av defekter och rumsligt varierande korrosion och visar hur probabilistisk modellering kan förbättra uppskattningarna av livslängden och optimera underhållsplaneringen. För armerade betongbroar införlivas övervakningsdata genom bayesiansk uppdatering, vilket minskar osäkerheten och synliggör möjligheten till en förlängning av livslängden som deterministiska bedömningar eller bedömningar baserade enbart på a priori-information inte kunde påvisa.

I båda fallen visar känslighetsanalyser och rumsliga analyser att inspektionsstrategier bör anpassas till nedbrytningens omfattning. Inledningsvis bör fokus ligga på de mest utnyttjade tvärsnitten, medan inspektionerna successivt bör utvidgas till att omfatta hela balken i takt med att nedbrytningen sprider sig. Resultaten stödjer därmed en adaptiv inspektionsplanering som först inriktas på högt belastade tvärsnitt och därefter gradvis utvidgas till andra områden allteftersom skadorna fortskrider.

En integrering av de redovisade tillförlitlighetsberäkningarna och resultaten från känslighetsanalyserna kan ge metodologiska, förvaltningsmässiga och samhälleliga

bidrag genom att främja en datadriven, kostnadseffektiv och hållbar broförvaltning, samtidigt som en säkerhetsnivå i överensstämmelse med internationella tillförlitlighetsstandarder upprätthålls.

Popular Science Summary

Concrete bridges and viaducts make up a significant portion of road infrastructure in Italy, Europe, and around the world. In the western world, many of them were built after the Second World War, when concrete construction expanded rapidly thanks to advantages such as partial prefabrication and the ability to create new structural shapes. During those decades, engineers also developed prestressing technologies, specifically *pretensioning* and *post-tensioning*, which involve tightening steel cables before or after the concrete is cast. These methods made it possible to build bridges with much longer spans, often exceeding 70 metres.

At the time, reinforced and prestressed concrete structures were widely believed to be almost eternal, often described as “liquid stone”, and the problem of steel corrosion inside concrete was not yet understood. However, several structural failures over the years revealed the weaknesses of these construction techniques, leading to improved design practices in modern bridge engineering.

Today, the urgent challenge is the assessment and management of thousands of existing ageing bridges. Their safety must be evaluated, their remaining service life estimated, and inspection and maintenance plans organised so that road networks remain functional. In Italy alone, there are more than 60,000 concrete bridges, making accurate and efficient assessment methods essential for sustainable and cost-effective infrastructure management.

Within this PhD project, a comprehensive framework is developed for evaluating the service life of concrete bridges affected by corrosion. Beyond the current state of research and engineering, it combines probabilistic structural performance modelling of the entire bridge with data from inspections, tests, and monitoring, allowing for building models that represent how real bridges deteriorate over time, accounting for empirical evidence.

The framework supports safety evaluation at multiple levels, from individual sections to entire bridge decks, and models corrosion using a combination of physical laws and evidence observed in existing structures. Two case studies, one on a post-tensioned bridge and one on a reinforced concrete bridge, demonstrate how the framework can estimate degradation and reveal the potential for extending service life beyond what traditional engineering approaches suggest.

For post-tensioned concrete bridges, the method captures the presence of defects and the spatially variable nature of corrosion, showing how probabilistic modelling improves service-life predictions and maintenance planning. For reinforced concrete bridges, integrating real data significantly reduces uncertainty and highlights the possibility of safe service-life extension, something that purely theoretical assessments often cannot demonstrate.

The results also show that inspection strategies should evolve as degradation progresses. At early stages, inspections should focus on the portions of the structure carrying the highest loads. As deterioration spreads, however, attention must gradually extend to the entire girder. This supports adaptive inspection planning that evolves with the bridge condition.

From a management perspective, the practical implications are substantial. The framework enables a safe extension of bridge service life, helping authorities postpone costly interventions, reduce environmental impacts, and focus resources where they are needed most. It also identifies which variables are most important to monitor at each stage of degradation, guiding more targeted and efficient inspection programmes.

Finally, beyond technical results, the work carries clear societal benefits: greater economic efficiency, improved sustainability, and enhanced public safety.

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Abbreviations

ALARP	As Low As Reasonably Practicable
ASCE	American Society of Civil Engineers
CC	Consequence class
CDF	Cumulative Distribution Function
CEN	European Standardisation Body
CO ₂	Carbon Dioxide
DS	Daniels System
FHWA	Federal Highway Administration
GPR	Ground Penetrating Radar
JCSS	Joint Committee of Structural Safety
LSF	Limit State Function
MLSC	Marginal Life-Saving Costs
NDT	Non Destructive Test
PDF	Probability Density Function
PMF	Probability Mass Function
PoD	Probability of Detection
PT	Post-Tensioned
RC	Reinforced Concrete
RF	Random Field
SCPS	Simplified Model for Corroded Prestressing Strands
RQ	Research Question
UT	Ultrasonic Tomography
WIM	Weigh-In-Motion

List of symbols

$[CO_2]$	CO ₂ content in the surrounding air	[-]
a	Pit depth	[mm]
A	Ageing function	[-]
a_A	Ageing factor	[-]
α	Pitting depth factor	[-]
A_s	Residual area of a reinforcement	[mm ²]
A_{sw}	Residual area of a stirrup	[mm ²]
b	Slab section width	[mm]
b_e	Activation energy	[kJ/mol]
b_w	Web width	[mm]
$b_{w,eff}$	Web width after spalling	[mm]
β	Annual reliability index	[-]
β_t	Annual target reliability index	[-]
C_0	Initial chloride concentration	[wt.-%/c]
c_c	Concrete cover	[mm]
C_{cr}	Critical chloride threshold	[wt.-%/c]
C_f	Capacity corresponding to the f-th failure mode	
c_i	Void classes	[-]
C_l	Clinker content	[kg/m ³]
Cl_{mo}	Chloride concentration model	[wt.-%/c]
$C_{s,app}$	Apparent surface chloride concentration	[wt.-%/c]
$C_{v,data}$	Number of detected voids	[-]
$C_{v,data}^{adj}$	Adjusted number of detected voids	[-]
c_w	Stirrup concrete cover	[mm]
d_0	Wire initial diameter	[mm]
D_0	Initial diameter of a reinforcement	[mm]
D_{app}	Apparent chloride diffusion coefficient	[mm ² /year]
Δd_c	Depth of the convection zone	[mm]
D_f	Demand corresponding to the f-th failure mode	
d_l	inner lever arm	[mm]
d_r	Carbonation depth from the r-th test	[mm]
$D_{RCM,0}$	Chloride diffusion coefficient	[mm ² /year]
d_s	Damage size	[cm]
EC	Exposure class	[-]
E_p	Young's Modulus of a prestressing wire	[MPa]
E_s	Young's Modulus of a reinforcement	[MPa]
ε_{p0}	Yielding strain deformation of a wire	[-]
ε_{pu}	Ultimate strain deformation of a wire	[-]
$\varepsilon_{pu,cor}$	Ultimate strain deformation of a corroded wire	[-]

ε_{su}	Ultimate strain deformation of a reinforcement	[-]
ε_{suc}	Ultimate strain deformation of a corroded reinforcement	[-]
ε_{sy}	Yielding strain deformation of a reinforcement	[-]
ε_{sync}	Yielding strain deformation of a corroded reinforcement	[-]
$\boldsymbol{\eta}$	Vector of degradation stochastic parameters	
H	Degradation development function	[mm ²]
F_{bDS}	Deck failure event in brittle Daniels System configuration	[-]
f_c	Concrete compressive strength	[MPa]
f_{ck}	Concrete characteristic compressive strength	[MPa]
F_f	Failure event associated with the f-th failure mode	[-]
F_G	Failure event for a girder	[-]
f_{pu}	Ultimate stress of a wire	[MPa]
$f_{pu,cor}$	Ultimate stress of a corroded wire	[MPa]
F_S	Deck failure event in series system configuration	[-]
f'_X	Prior distribution of the random variable X	[-]
f''_X	Posterior distribution of the random variable X	[-]
f_y	Yielding stress of a wire	[MPa]
f_{sy}	Yielding stress of a reinforcement	[MPa]
f_{sync}	Yielding stress of a corroded reinforcement	[MPa]
f_{vw}	Yielding stress of stirrups	[MPa]
g_{bDS}	Deck limit state function in brittle Daniels System configuration	
g_c	Limit state function for corrosion activation	
g_f	Limit state function for the f-th failure mode	
g_G	Limit state function for a girder	
g_{HJ}	Limit state function for half joint failure mode	[kN]
g_M	Limit state function for bending failure mode	[kNm]
g_S	Deck limit state function in series system configuration	
g_{Sp}	Spalling limit state function	[year]
g_V	Limit state function for shear failure mode	[kN]
I	Indication event	[-]
i_c	Corrosion current density	[$\mu\text{A}/\text{cm}^2$]
k	Carbonation parameter	[mm ² /year ^{1/2}]
K_c	Model uncertainty in the chloride diffusion process	[-]
k_d	Carbonation parameter in dry conditions	[mm ² /year ^{1/2}]
k_e	Environmental regression parameter	[-]
k_t	Transfer parameter	[-]
k_S	Thickness of the strut	[mm]
k_w	Carbonation parameter in wet conditions	[mm ² /year ^{1/2}]
L	Likelihood function	[-]
L_{girder}	Girder length	[m]
M	Stochastic degradation model	
M^{-1}	Inverse of the stochastic degradation model	

M_C	Bending capacity	[kNm]
μ_{lim}	Limit corrosion degree of a wire	[-]
μ_{loss}	Corrosion degree of a wire	[-]
$\mu_{C_{s,ann}}$	Mean value of the apparent surface chloride concentration	[wt.-%/c]
M_D	Bending demand	[kNm]
n_{MCS}	Number of Monte Carlo simulations performed	[-]
$N_{D,S}$	Axial demand of the strut	[kN]
$N_{D,T}$	Axial demands of the tie	[kN]
N_S	Axial capacities of the strut	[kN]
N_T	Axial capacities of the tie	[kN]
O_j	Number of observed voids for girder j	[-]
O_i^{adj}	Adjusted number of observed voids for girder j	[-]
p'	Prior distribution of a parameter	[-]
p''	Posterior distribution of a parameter	[-]
$P_{\mathbb{D}}$	Probability of defect presence	[-]
P_{tot}	Prestressing force applied to a tendon	[kN]
$p_{v,anna}^{adj}$	Adjusted probability mass function of the void size	[-]
ϕ_w	Stirrup diameter	[mm]
Φ^{-1}	Inverse cumulative distribution function of the Standard Gaussian Distribution	
$\overline{PoD}_{c_i}$	Average probability of detection of voids within class c_i	[-]
q_1	Distributed traffic load on lane 1	[kN/m ²]
Q_1	Tandem traffic load on lane 1	[kN]
q_2	Distributed traffic load on lane 2	[kN/m ²]
Q_2	Tandem traffic load on lane 2	[kN]
q_3	Distributed traffic load on lane 3	[kN/m ²]
Q_3	Tandem traffic load on lane 3	[kN]
q_a	Linear permanent load	[kN/m]
r	Annual reduction in bar radius	[mm]
r_0	Initial wire radius	[mm]
RH	Relative humidity	[%]
ρ	Spearman correlation coefficient	[-]
$\rho_C \mathbb{D}$	Corrosion spot occurrence rate variable	[1/m]
$\rho_{CV} \mathbb{D}$	Joint corrosion spot and void occurrence variable	[1/m]
$\rho_V \mathbb{D}$	Void occurrence rate variable	[1/m]
R_{tot}	Total resistance of a tendon	[kN]
$\mathbf{S}$	Vector of the Demand-related input variables	
S_{bDS}	Deck survival event in brittle Daniels System configuration	[-]
S_C	Vector containing corrosion spot positions	
S_{CV}	Vector containing corrosion spot in the presence of a void positions	
Σ_p	Covariance matrix of the parameters	
$\sigma_{w,max}$	Maximum stress capacity of a wire	[MPa]

S_V	Vector containing void positions	
S_f	Survival event associated with the f-th failure mode	[-]
S_G	Survival event for a girder	[-]
S_S	Deck survival event in series system configuration	[-]
t	Time variable	[year]
t_A	Time of the assessment	[year]
T_H	Time needed for the degradation process to become active	[year]
T_q	Time to generalised corrosion	[year]
$T_{q p}$	Time to generalised or pitting corrosion	[year]
Θ	Vector of a distribution parameters	
θ_d	Inner arm geometric uncertainties	[-]
$\theta_{f,C}$	Capacity model uncertainties associated with the f-th failure model	[-]
$\theta_{f,D}$	Demand model uncertainties associated with the f-th failure model	[-]
θ_S	Slab thickness geometric uncertainties	[-]
ToW	Time of Wetness	[-]
T_p	Time to pitting corrosion	[year]
T_{real}	Air temperature	[k]
T_{ref}	Model reference air temperature	[k]
U_{meas}	Measurement uncertainty variable	[-]
u_w	stirrup spacing	[mm]
V_C	Shear capacity	[kN]
V_D	Shear demand	[kN]
V_P	vertical component of the prestressing force	[kN]
V_{corr}	Corrosion rate	[mm/year]
$V_{corr,a}$	Active corrosion rate	[mm/year]
$\mathbf{X}$	Vectors of the Capacity-related input variables	
$x_{k_{975}}$	Characteristic value with return period equal to 975 years	[-]
$x_{k_{T_R}}$	Characteristic value with return period equal to T_R years	[-]
ξ_s	Corrosion degree of a reinforcement	[-]
y_c	Position of the neutral axis	[mm]
$\zeta_C \mathbb{D}$	Corrosion spot spatial distribution variable	[-]
$\zeta_{CV} \mathbb{D}$	Joint corrosion spot and void spatial distribution variable	[-]
z	Inner lever arm of the stirrups	[mm]
$\zeta_V \mathbb{D}$	Void spatial distribution variable	[-]

Publications

Appended Papers

- I. S. Celati, A. Natali, W. Salvatore, I. Björnsson, and S. Thöns, ‘Spatial and time-dependent reliability analysis for post-tensioned concrete decks subjected to multiple failure modes’, *Struct. Saf.*, vol. 117, p. 102634, Nov. 2025, doi: 10.1016/j.strusafe.2025.102634.
- II. Mazzatura, W. Salvatore, S. Caprili, S. Celati, M. Mori, and M. Gammino, ‘Damage detection, localization, and quantification for steel cables of post-tensioned bridge decks’, *Structures*, vol. 57, p. 105314, Nov. 2023, doi: 10.1016/j.istruc.2023.105314.
- III. S. Celati, F. Mattei, W. Salvatore, and S. Thöns, ‘Statistical Data Analysis of Inspection Outcomes of Post-Tensioned Concrete Bridges in Italy’, in *IABSE Congress Ghent 2025*, Ghent, Belgium, Aug. 2025, pp. 108–115. doi: 10.2749/ghent.2025.0108.
- IV. S. Celati, F. Mattei, W. Salvatore, and S. Thöns, ‘Reliability analysis of post-tensioned concrete bridges using inspection-based modelling of grout injection voids’, *Struct. Infrastruct. Eng.*, pp. 1–20, Apr. 2026, doi: 10.1080/15732479.2026.2651979.
- V. Simone Celati, Agnese Natali, Walter Salvatore, Sebastian Thöns: ‘System Reliability Analysis of Reinforced Concrete Bridges Using Multiple Data Sources’ (Ready for submission)

Author’s contribution to appended papers

- I. The Author developed the approach adopted for the reliability analysis and the associated probabilistic models. The Author also performed all analyses, wrote the manuscript, and revised it based on feedback from the co-authors.
- II. The Author contributed to the conceptualisation of the inspection methodology, and designed and carried out the in-situ campaigns used to test and calibrate the procedure. The Author also participated in the analysis of the inspection outcomes and provided feedback to the first author.
- III. The Author collected and analysed the complete dataset, wrote the manuscript, and revised it following feedback from the co-authors.
- IV. The Author developed the probabilistic models and advanced simulation procedures used to perform the reliability analyses, and executed all computational tasks. The manuscript was written and revised by the Author in collaboration with the co-authors.
- V. The Author developed the approach adopted for the reliability analysis, the associated probabilistic models, and the updating procedures introduced in the study. Moreover, the Author acquired part of the input data, analysed

the entire dataset, and wrote and revised the manuscript based on co-author feedback.

Other related publications

Scientific contributions not included in the present thesis

- a) S. Celati, A. Natali, and W. Salvatore, ‘Traffic models for Italian provincial roads’, in *The Essence of Structural Engineering for Society*, Ghent, Belgium, Aug. 2025, pp. 366–374. doi: 10.2749/ghent.2025.0366.
- b) F. Del Carlo, S. Celati, and S. Caprili, ‘The Impact of Climate Change Scenarios on The Time-Dependent Capacity of Reinforced Concrete Structure’, presented at the IABSE Symposium, Tokyo 2025: Environmentally Friendly Technologies and Structures: Focusing on Sustainable Approaches, Tokyo, Japan, 2025, pp. 388–396. doi: 10.2749/tokyo.2025.0388.
- c) F. Del Carlo, S. Celati, and S. Caprili, ‘Time-dependent Fragility Curves for the Seismic Risk Analysis of Reinforced Concrete Buildings subjected to Degradation’, presented at the IABSE Symposium, Tokyo 2025: Environmentally Friendly Technologies and Structures: Focusing on Sustainable Approaches, Tokyo, Japan, 2025, pp. 1819–1827. doi: 10.2749/tokyo.2025.1819.
- d) A. Natali, S. Celati, D. Gaudioso, W. Salvatore, D. Coletta, and M. Malavisi, ‘The knowledge process for the structural assessment of existing concrete bridges with post-tensioned cables: Application to a case study’, in *Bridge Maintenance, Safety, Management, Digitalization and Sustainability*, 1st edn, London: CRC Press, 2024, pp. 2645–2653. doi: 10.1201/9781003483755-315.
- e) I. Björnsson, S. Thöns, S. Celati, and B. Hergart, ‘Towards a service life extension of the Øresund Fixed Link’, in *Bridge Maintenance, Safety, Management, Digitalization and Sustainability*, 1st edn, London: CRC Press, 2024, pp. 3133–3141. doi: 10.1201/9781003483755-372.
- f) S. Celati, A. Natali, W. Salvatore, and S. Thöns, ‘System reliability accounting for corrosion-induced degradation over time’, presented at the IABSE Symposium Manchester 2024: Construction’s Role for a World in Emergency, Manchester, United Kingdom: International Association for Bridge and Structural Engineering, 2024, pp. 655–663. doi: 10.2749/manchester.2024.0655.
- g) S. Celati, A. Natali, W. Salvatore, and S. Thöns, ‘Temporal reliability evolution of a prestressed girder with post-tensioned tendons subjected to corrosion’, *Procedia Struct. Integr.*, vol. 62, pp. 361–368, 2024, doi: 10.1016/j.prostr.2024.09.053.

- h) S. Thöns, I. Björnsson, and S. Celati, ‘On service life quantification and service life management’, *Procedia Struct. Integr.*, vol. 62, pp. 259–267, 2024, doi: 10.1016/j.prostr.2024.09.041.
- i) S. Celati et al., ‘Operational Modal Analysis as a Tool for Bridge Model Updating. Application to an Unconventional Case Study’, in *Experimental Vibration Analysis for Civil Engineering Structures*, vol. 432, M. P. Limongelli, P. F. Giordano, S. Quqa, C. Gentile, and A. Cigada, Eds, in *Lecture Notes in Civil Engineering*, vol. 432. , Cham: Springer Nature Switzerland, 2023, pp. 1–10. doi: 10.1007/978-3-031-39109-5_1.

1 Thesis overview

1.1 Background

Concrete bridges are fundamental components of the infrastructure network that sustain our modern society [1], [2]. Their functionality is generally taken for granted, yet any malfunction can lead to significant direct and indirect costs, while also provoking fear, anger, and distress, and thereby impose substantial social burdens [3].

A significant number of concrete bridges in Europe and worldwide is ageing, placing increasing demands on structural engineers to assess their condition and manage their performance. The collapse of several reinforced concrete bridges (RC) or prestressed concrete bridges with post tensioned tendons (PT) bridges in Italy (e.g., Petrulla Viaducts, 2014 [4], Fossano bridge, 2017 [4], Polcevera Bridge, 2018 [5]) and elsewhere in Europe (see, for instance, Carola Bridge, 2024) underscores the challenges and urgency of bridge management and condition assessment. At the same time, the global drive for sustainability and cost efficiency puts infrastructure management authorities under pressure to extend the service life of existing structures [6]. For example, ASCE estimates that \$123 billion are needed to repair the deteriorating US bridges, as reported in the 2017 Infrastructure Report Card. Instead, extending service life helps avoid costly interventions and replacements, thereby reducing greenhouse gas emissions and optimising financial resources [7]. Balancing these two seemingly opposing needs, ensuring the safe management of existing structures on the one hand and avoiding unnecessary resource waste on the other, requires an accurate assessment of how the safety of structural components evolves over time.

Indeed, bridges often operate beyond their intended service life due to their difficult and costly replacement, highlighting the need for the development of a rational and relevant assessment framework capable of estimating and predicting bridge degradation and informing decision-makers.

From a normative perspective, several national guidelines have been issued on bridge inspection, assessment, and maintenance, such as the *US Manual for Bridge Evaluation* [8], the *Italian Guidelines for Risk Classification and Management, Safety Assessment, and Monitoring of Existing Bridges* [9], and the *Swedish Code for Assessment of Existing Bridges* [10]. At an international level, guidelines that

explicitly account for uncertainties in the assessment of existing structures are provided, for example, in ISO 2394:2015 [11] and, with a focus on concrete structures, in *fib Bulletin 80* [12]. Taken together, these documents recognise the importance of safety assessment for existing bridges and address it through reliability analysis combined with evaluation of the conservation state.

However, several open challenges remain for efficient and sustainable infrastructure management. These include estimating residual service life, prioritising bridges within large inventories, identifying critical variables for testing, designing and executing inspection and testing campaigns, and integrating newly acquired knowledge from monitoring and testing into reliability assessments. These challenges motivate and guide the present research, which seeks to address them systematically throughout this thesis.

1.2 Research questions

The general objective of this work is to develop and propose a new framework for estimating the service life of concrete bridges, with particular attention to corrosion-induced degradation, directly applicable to the management of existing bridge inventories. The framework is developed within the context of system reliability analysis, which allows the explicit consideration of the uncertainties characterising structural assessments and enables the evaluation of the overall performance of the structural system. Moreover, it aims to support decision-making strategies for bridge prioritisation, assessment, inspection and testing design, as well as for the continuous updating of structural knowledge.

This broad research objective can be divided into several explicit research questions, which guided the research work and the dissertation development:

- 1) How can system reliability analyses be performed to account for the temporal and spatial evolution of degradation, as well as the interactions among failure modes of structural elements?
 - a. What are the main factors influencing service-life estimation in existing RC and PT bridges?
- 2) How can the effects of different concrete quality and conservation status on the degradation process be integrated into the system reliability model?
- 3) Regarding PT bridges, how can the influence of poor duct injection on the service-life estimation of PT bridges be modelled?
 - a. How can inspections of PT cables be designed to detect grout voids and corrosion within ducts?

- b. How are defects within injected ducts distributed? Is it possible to probabilistically describe the relation between the presence of voids and the presence of corrosion?
 - c. How to perform a system reliability analysis of PT girders accounting for the presence of defects within ducts?
- 4) How can knowledge acquired through inspection and monitoring campaigns be incorporated into system reliability analyses to update models and refine the service-life estimation of bridges?
 - a. How does different spatial modelling affect the system reliability estimates?

The novelty of this thesis lies in the development of a comprehensive system reliability framework that explicitly accounts for spatial and temporal degradation, multiple interacting failure modes, and Bayesian updating. This combination goes beyond existing approaches, which are often limited to component-level analyses or rely on simplified degradation assumptions.

The research questions are addressed through a combination of probabilistic modelling techniques and in situ inspection and testing campaigns. Figure 1-1 illustrates the overall research scheme together with the research questions outlined and the research papers that gather the work and outputs of each research step.

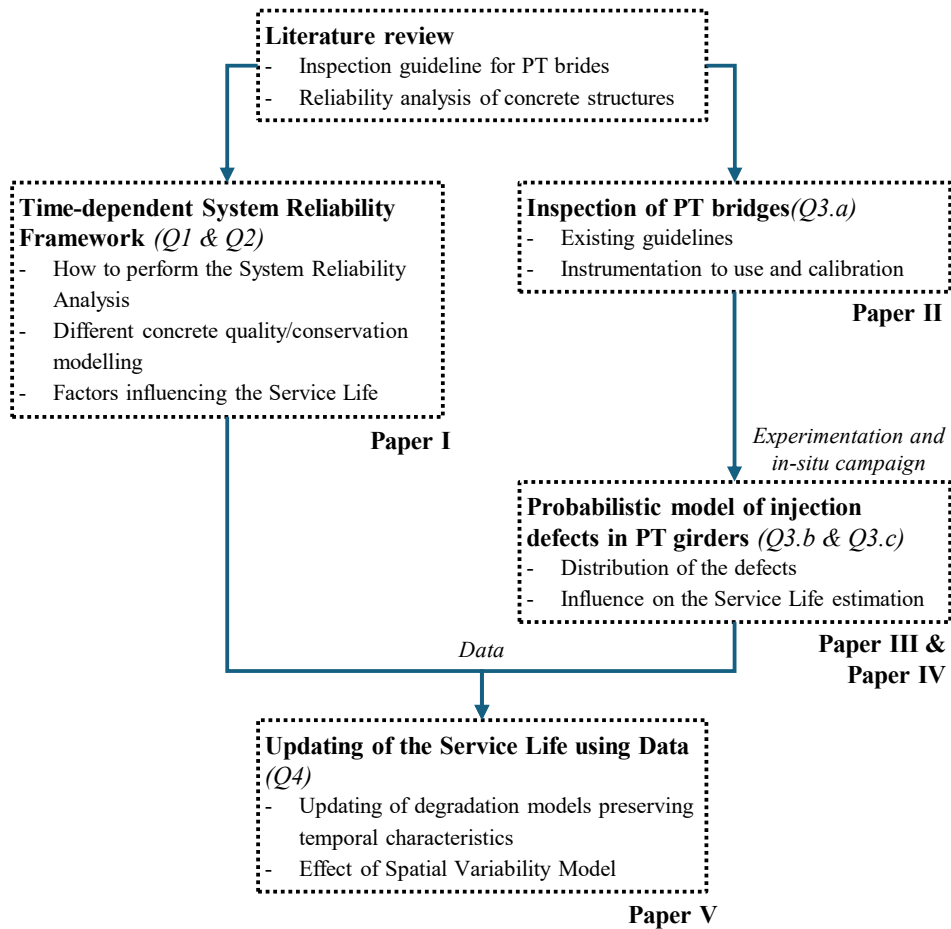


Figure 1-1: Visualisation of the Research scheme

1.3 Outline of the thesis

The research papers developed during the Ph.D. are organised into eleven chapters and included as appendices to this dissertation. The principal methodologies and results from these papers are incorporated within each chapter.

Chapter 2 presents the state of research in system reliability analysis. The breadth of the subject is illustrated through a bibliometric analysis, and relevant methods and literature are described in several Sections. The requirements imposed on structures by modern society, as described in standards and guidelines, are eventually presented. These requirements combine economic considerations with life-safety concerns in relation to the consequences of structural failure. Target

reliability indices are defined as minimum thresholds for the safe and efficient operation of a structure.

Based on the research gaps identified in Chapter 2, Chapter 3 provides the objectives and illustrates the research process pursued throughout the thesis, along with the specific methods utilised. Based on these objectives and methods, the system reliability assessment framework is developed.

Chapter 4 introduces a novel comprehensive reliability framework for estimating the system reliability of concrete bridges. The framework incorporates spatial and time-dependent analyses of corrosion degradation and defines system limit state functions that account for multiple failure modes. Methods for Bayesian updating of random variables are also presented.

Focusing on PT bridges, Chapter 5 begins with an inspection campaign designed to gather data on cable conservation. The collected data are analysed to develop a probabilistic model for characterising injection voids and corrosion spots within PT ducts.

In Chapter 6, the probabilistic model for predicting cable conservation status in PT bridges is integrated into the system reliability framework. In particular, a novel procedure for calculating the spatial and time-dependent system reliability, accounting for the random positions and numbers of injection voids and corrosion spots, is developed and presented.

Chapter 7 focuses on corrosion degradation. It first introduces models for corrosion initiation and, building on recent laboratory and field studies, proposes a novel probabilistic model for corrosion triggering, grounded in logical system modelling to account for different concrete conditions. It then presents corrosion progression models, concluding with specific models for prestressing wires in PT bridges.

In Chapter 8, the developed reliability framework is applied to a PT deck and an RC deck, highlighting the effects of different concrete conditions, system modelling (including multiple failure modes), and variable updating on service-life estimation. A sensitivity analysis of key variables is also presented, providing insights for the design of future inspection campaigns.

Conclusions are provided in Chapter 9, which summarises the main findings of the research, discusses their significance, and reflects on the fulfilment of the research aims and objectives, as well as the answers to the research questions.

Chapter 10 discusses paths for future research. Suggestions for further investigation are outlined to address current limitations and guide improvements in understanding structural reliability and service life.

Chapter 11 concludes the thesis by briefly summarising the appended papers and their key findings, providing a concise overview of the Ph.D. project's journey and outcomes.

2 State of research in System Reliability Analysis of Existing Concrete Bridges

A state-of-the-research analysis is conducted to examine the main methods and calculation techniques used in system reliability assessment of existing concrete structures subjected to degradation. The purpose is to delineate the research domain and identify existing gaps in the current literature. The Chapter begins by presenting the results of a bibliometric analysis aimed at clarifying the breadth of the subject (Section 2.1). It then reviews the main methods and literature on system reliability analysis, time-dependent reliability, and Bayesian updating in structural reliability (Section 2.2), followed by an overview of the state of research on reliability of concrete structures and corrosion in concrete bridges (Section 2.3). Finally, Section 2.4 discusses the reliability requirements for both newly designed and existing civil structures, and Section 2.5 summarises the identified gaps and the scope of the thesis.

2.1 Bibliometric Analysis

A bibliometric analysis [13] is carried out using the Scopus database, which allows an extensive publication coverage, to investigate thematic trends across four research domains: existing concrete bridges, corrosion in concrete structures, reliability analyses, and bridge inspection and monitoring. In particular, a keyword co-occurrence analysis is performed to explore the relationships among these domains and to identify research trends.

Four search queries, i.e., one for each domain, are defined and combined pairwise:

- 1) Reinforced Concrete OR post-tensioned concrete OR concrete bridges OR existing concrete bridges;
- 2) Reinforcement corrosion OR concrete deterioration OR cables corrosion OR chloride induced corrosion OR concrete carbonation OR concrete durability;

- 3) Structural reliability OR system reliability OR probabilistic modelling OR Bayesian analysis;
- 4) Inspection OR Monitoring OR Data gathering OR maintenance planning.

These queries are refined using wildcards (*) to capture word variants (e.g., model, models, modelling). Triplet or quadruple combinations are not considered, as the resulting number of publications is insufficient for meaningful bibliometric analysis.

The six pairwise datasets are merged into a single dataset, which serves as the basis of the analysis. The breadth of this research domain is summarised in Figure 2-1, which shows a keyword co-occurrence network. To enhance readability, only the forty most frequent keywords are displayed. In the figure, keywords are grouped into clusters (distinguished by colour) based on their co-occurrence, while link thickness reflects the strength of association between terms. “Reinforced concrete” emerges as the most central term, acting as a bridge among different thematic areas. Three main clusters can be identified: the red cluster (top-left) concerns reinforced concrete structures and bridges, and corrosion. It is closely related to concrete structural analysis and inspection, particularly in connection with decision-making. The green cluster (bottom) emphasises the treatment of uncertainties through reliability and risk analyses, with time-dependent aspects highlighted by terms such as life-cycle and corrosion effects. Notably, no keywords explicitly related to “system reliability” appear, indicating a limited literature output in this area. The third cluster (blue, top-right) focuses on durability, maintenance, and bridge management. A preference for coastal structures is evident, as reflected in the use of chloride-induced and seawater corrosion keywords, combined with maintenance cost constraints. However, keywords related to defect detection or probabilistic modelling of PT bridges are absent, suggesting a lack of targeted research.

The clustering highlights a clear imbalance in the research domain. Reliability studies are closely linked to corrosion, degradation, maintenance, and reinforced concrete structures, but they rarely address deterioration in PT bridges. Keywords specifically linked to injection defects or tendon corrosion are absent, indicating a lack of targeted investigations. Moreover, while durability and management aspects are frequently addressed, they are often treated independently of system reliability concepts.

Instead, this research work spans all three identified clusters by integrating mechanical models for the assessment of concrete structures with uncertainty treatment via system reliability analysis. Within this framework, corrosion initiation and propagation are modelled probabilistically, explicitly accounting for both chloride concentration and alkalinity reduction. Finally, aspects of maintenance, management, and service-life estimation are addressed through the definition of target reliabilities based on consequence modelling and supported by data-driven updating procedures.

The research questions (RQs) defined in Section 1.2 guide the organisation of this state-of-research review. In particular, the Chapter presents the core concepts underpinning system reliability analysis, including logical system modelling, spatial dependency, time-variant reliability analysis, parameter sensitivity, and Bayesian updating techniques.

Building on these principles, the Chapter proceeds to analyse the state of research concerning the application of system reliability analysis to concrete bridges, with specific attention to RC and PT structures and their characteristic degradation mechanisms. It then discusses the reliability targets adopted in standards and guidelines to support service-life estimation following system reliability calculations. Finally, it concludes with a synthesis of the main research gaps identified through this analysis and outlines the specific objectives addressed in this thesis

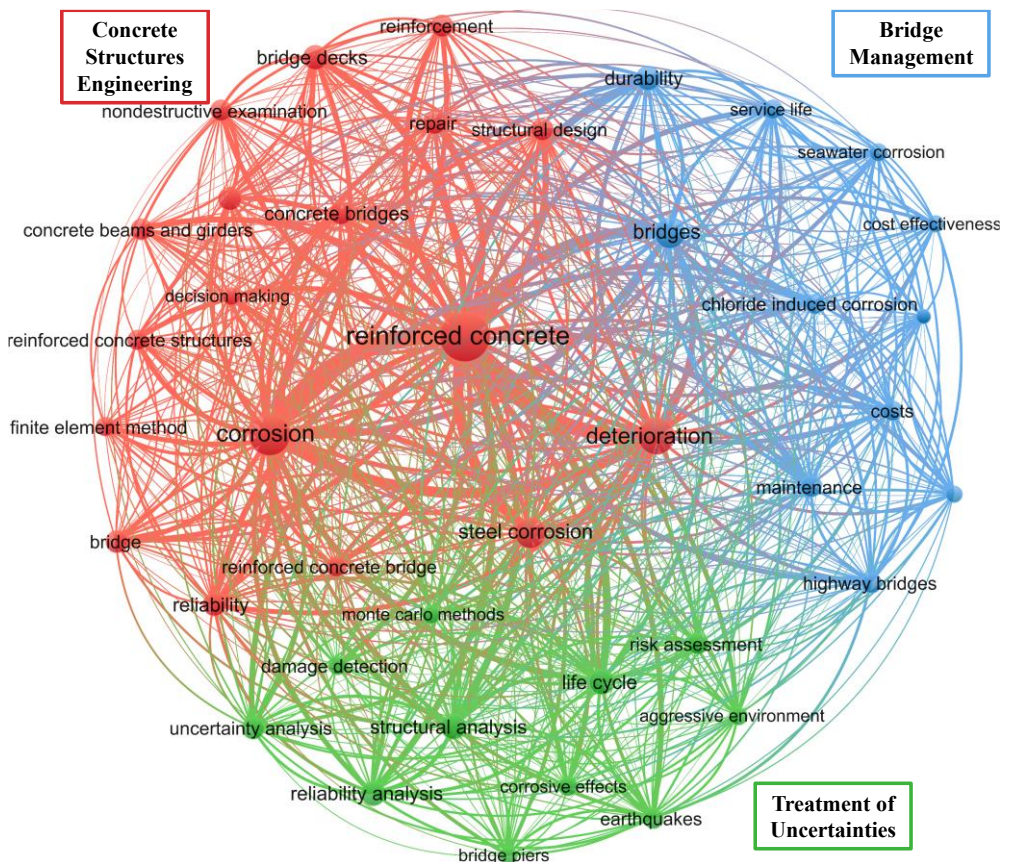


Figure 2-1: Bibliometric analysis of research publications keywords related to existing RC or PT bridges and reliability analysis, (VOSviewer, [14]).

2.2 System Reliability analysis

Since the early 2000s, the potential of probabilistic methods in assessing existing structures has been increasingly recognised. For example, the *Assessment of Existing Structures* guideline was published in 2001 by the Joint Committee of Structural Safety (JCSS) [15]. Focusing on bridges, the Danish Road Administration introduced a standard for road bridge classification based on probabilistic assessments [16], followed shortly by Switzerland and the Netherlands, which issued their own standards [17], [18]. At the European level, reliability analyses are recognised in the Eurocodes as a valid method for assessing existing structures, and the CEN (European Standardisation Body) is currently working to unify probabilistic methods for this purpose. These developments reflect a growing consensus that probabilistic approaches enable a more rational treatment of uncertainty and time-dependent deterioration, making them particularly suitable for long-term performance assessment. Importantly, they also allow global (system-level) evaluations that overcome the limitations of purely sectional (local) analyses.

System reliability analyses have long recognised that bridges cannot be assessed as isolated components, since failure arises from the interaction of multiple elements and modes. Classical formulations, series, parallel, or hybrid provide useful idealisations, but the mechanical interpretation of parallel systems for real structures is limited [19]. In practice, redistribution after element failure governs the system response, with brittle and ductile assumptions framing two extreme cases: *ideal elastic-brittle* and *ideal elastic-plastic*. In the first case, an element behaves elastically until reaching its maximum capacity, after which it loses all residual strength. In the second case, once the maximum capacity is reached, the element exhibits ductile behaviour and can continue to sustain loads at a constant maximum capacity level.

In principle, assessing the probability of system failure requires examining all possible failure sequences within a structure. However, exhaustive evaluation is computationally infeasible. Consequently, research has prioritised the identification of dominant failure paths (see, for example, [20], [21]) or the adoption of simplified system models. Daniels Systems have been employed to represent progressive failure governed by the weakest elements. This approach enables analytical formulation of capacity estimates in both brittle and ductile cases [19], [22], facilitating the solution of the system reliability problem.

Modelling the correlation among structural components represents a significant challenge. However, studies indicate that neglecting such correlation can lead to significant discrepancies in system reliability estimates and the failure mode of the structures [19], [23], [24], [25]. Component correlations may arise from modelling assumptions, such as shared uncertainties in loads or material properties, as well as from spatial variability in concrete or reinforcement characteristics. Copula-based

approaches have been proposed to capture dependencies when a joint distribution model for the input variables is unavailable. These approaches combine individual marginal distributions and account for their interdependence through a Copula function, which defines the joint distribution dependence structure [26]. Several Copula families exist, and the most suitable one can be selected using goodness-of-fit tests (see, for instance, [27], [28]).

Spatial variability, on the other hand, can be described as a function of the distance between components. Spatially dependent variables are typically represented by random fields (RFs) [29], with the correlation function type and correlation length defined according to relevant studies, e.g., [30], [31]. When the marginal distribution of the RF is non-Gaussian, a translation random field can be used. By mapping a Gaussian random field through a monotonic increasing function, $f : \mathbb{R} \rightarrow \mathbb{R}$, defined as an isoprobabilistic transform, the required non-Gaussian marginal distribution can be obtained [32].

Therefore, both approaches effectively model complex dependencies and can also be used jointly to represent spatially variable correlated variables, i.e., multiple variables that are mutually correlated and simultaneously vary in space, as demonstrated in [33].

2.2.1 Time-dependent system reliability

Research on structural systems has developed various formulations to capture failure dependencies, yet most of these remain time-invariant. However, real structures deteriorate over time, making time-dependent reliability analyses essential for describing the evolution of safety throughout the service life.

Time dependency is typically modelled by introducing random processes that describe the progression of degradation or variable loads. Two types of failure events are usually considered: point-in-time failure and cumulative failure. In the first case, failure may occur at any time during the considered period, and its probability has often been used to describe time-dependent reliability (e.g., [34], [35]). This approach, while straightforward, only evaluates the structural condition at a specific instant, without accounting for the possibility that failure may have already occurred. In contrast, the cumulative failure event considers that failure may happen at any instant within a given interval, thereby incorporating the structural history up to the considered time. The probability associated with this event can then be used to determine the cumulative reliability of the structure.

Despite being more rational, cumulative reliability calculations are challenging [36] and strongly depends on the correlation of failure events over time. For the typical case where capacity-related variables can be separated from demand-related variables, the time-dependent reliability analysis needed to calculate cumulative reliability can be reduced to multiple time-independent analyses. In this

formulation, the total time span is divided into several intervals (often one year each), and the cumulative failure event is modelled as the union of all point-in-time failures, thereby transforming the time-variant problem into a time-invariant series-system problem. The probability of each point-in-time failure can be approximated by considering the minimum capacity and the maximum demand within the corresponding interval.

Moreover, bounds for the cumulative failure probability can be derived based on the assumed temporal dependency among point-in-time failures. In the case of full dependency, the cumulative probability equals the maximum point-in-time probability; in the case of independence, it equals the sum of the individual probabilities [37]. These bounds are generally wide, and tighter estimates can only be obtained by explicitly modelling the temporal correlation of failure events (see [37], [38]).

2.2.2 Sensitivity Analysis and Bayesian Updating in Structural Reliability

Information on the Capacity and Demand of structural systems can be obtained through inspection and monitoring campaigns, and their integration into a system reliability framework improves the relevance of the failure probability estimation, as shown in [39].

A key challenge, however, lies in deciding which variables should be tested. In practice, it is rarely known in advance how updating a particular variable will influence the overall reliability assessment. For this reason, sensitivity analyses are essential tools for directing inspection and monitoring resources. Sensitivity measures allow researchers to identify which variables most strongly influence reliability estimates, thereby guiding data collection. Among various techniques (see, for instance, [40]), rank-based correlation indicators, such as the Spearman coefficient (ρ), have been widely used due to their robustness and limited assumptions. These coefficients quantify how the uncertainty associated with an input variable contributes to the overall uncertainty of the limit state function (LSF). Consequently, reducing the uncertainty of variables with a high (in absolute value) Spearman coefficient leads to a substantial reduction in the uncertainty of the LSF and, thus, to more relevant system reliability estimates. Moreover, the sign of the coefficient provides additional insight: a positive value of ρ indicates that increasing the variable increases the LSF; vice versa, a negative value indicates that increasing the variable decreases the LSF.

Once relevant variables are identified, Bayesian analysis offers a systematic way to integrate inspection and monitoring results into probabilistic models. This approach has been extensively applied in structural reliability (e.g., [41], [42], [43]) to refine prior models using empirical data, whether through direct updating of measurable

parameters (e.g., concrete strength, steel yield stress) or indirect updating of degradation models (e.g., carbonation or chloride ingress) [43].

In general, the outcome of an inspection or monitoring activity is modelled as a realisation of a random variable, linked to the input variable via a likelihood function. This likelihood function expresses the probability of observing the test outcome given a specific realisation of the input variable(s). It therefore incorporates the information provided by real-world evidence into the prior probabilistic model of the input variables.

Bayes' theorem provides the mathematical tool for updating and estimating the posterior distribution of the input variables. However, because the calculations require the integration of products of probability density functions, analytical solutions rarely exist. Therefore, numerical integration or simulation-based procedures, such as Markov Chain Monte Carlo methods or Bayesian updating with structural reliability methods [42], are typically adopted.

Depending on the type of measurement, the obtained information can be classified as equality or inequality information [44]. In the first case, the test outcome is a numerical value, and the likelihood function of the observation can be defined based on the measurement error probability density function, either assumed or modelled according to the instrumentation used. In the second case, the information refers to exceeding a certain measured threshold [45], [46], e.g., presence of corrosion. Here, the likelihood function must be defined using a probability mass function that represents the discrete outcome space of the test result (see [44]).

Finally, Bayesian updating can also be employed to update the calculated failure probability directly, and thus the reliability of a structure, based on information such as its survival of extreme events or its continued survival over time [47], [48].

Overall, the combination of sensitivity analysis and Bayesian updating provides an effective pathway to incorporate real-world evidence into probabilistic models, thereby enhancing the relevance of structural reliability calculations for existing concrete structures.

2.3 System Reliability Analysis of Concrete bridges

In the previous Section, concepts regarding logical system modelling, spatial and time dependencies, sensitivity analysis and Bayesian updating are presented. These are widely used for the spatial- and time-dependent reliability assessment of existing concrete bridges, as conventional methods may underestimate the actual structural reliability [49]. For instance, regarding RC bridges, Jacinto et al. [50] analysed a corroded bridge and updated probabilistic calculations using in situ data; Vishwanath and Banerjee [51] proposed an approach to assess the seismic

vulnerability of corroded concrete piers; and Liljefors and Köhler [52] developed a decision-support tool based on structural assessment, focusing on shear failure reliability of a bridge section. While these studies demonstrate the feasibility of applying probabilistic methods to deteriorated RC structures, they also reveal important shortcomings. Jacinto et al. [50] did not model the time-dependent progression of corrosion, considering only a single reference year and focusing solely on the critical section where corrosion was visible. This simplification neglects both the temporal evolution of corrosion and its spatial variability, leading to an overestimation of current reliability and hindering service-life predictions. In contrast, Vishwanath and Banerjee [51] and Liljefors and Köhler [52] concentrated on individual components without accounting for the overall structural system behaviour. By omitting system-level interactions, their approaches may overestimate structural reliability and service life, potentially leading to sub-optimal maintenance and intervention strategies.

For prestressed concrete bridges with post-tensioned tendons (PT bridges), the literature has addressed additional complexities related to tendon deterioration and defective grouting. Guo et al. [53] performed a time-dependent reliability analysis accounting for the rheological properties of concrete and steel, as well as the potential presence of existing damage. Pillai et al. [54], building on extensive experimental work with various void configurations [55], developed a probabilistic model that explicitly accounted for voids within ducts and external tendon damage and employed it to perform reliability analyses. Nguyen et al. [56] modelled voids in injected grout and described pit occurrence in tendons using a Poisson distribution, but assumed voids were continuously present along the cable length, an assumption that led to service-life estimates consistently below 50 years. Al-Mosawe et al. [57] analysed the failed Ynys-y-Gwas bridge in the UK and conducted a reliability assessment that accounted for uncertainties in bridge geometry, tendon tension loss, and cable deterioration, based on post-failure inspection data. Together, these studies highlight progress in modelling tendon deterioration but also underscore persistent limitations: (i) degradation is often assumed uniform along tendons, with little attention to spatial variability; (ii) duct defect modelling remains overly simplified; and (iii) large-scale, high-quality inspection datasets are still lacking. These limitations reduce the relevance of reliability assessments, often resulting in overly conservative outcomes that may distort maintenance planning and resource allocation. Furthermore, under certain assumptions, reliability may be overestimated, increasing safety concerns. Both outcomes undermine sustainable asset management, either by wasting resources or by compromising structural safety.

Moreover, most existing studies focus either on individual component failures [52], [53], [58], [59], [60] or on system-level models [61], [62], [35], [63], [64], without fully accounting for the interactions among multiple failure modes within a single component. Neglecting such interactions can significantly bias reliability

predictions, as degradation mechanisms often develop in parallel and influence multiple failure modes.

2.3.1 Corrosion in Concrete bridges

Corrosion is widely recognised as one of the main degradation mechanisms affecting concrete bridges, both in academic literature and in normative documents (see, for instance, [12], [65], [66], [67], [68], [69], [70], [71], [72], [73]). Its impact is particularly severe in regions with harsh environmental conditions, such as fluctuating humidity, chloride exposure, or elevated carbon dioxide (CO₂) levels, where deterioration can quickly compromise safety. Among the aggressive agents, chlorides, commonly introduced through de-icing salts or marine environments, are especially harmful because they cause localised corrosion, a primary driver of reinforcement and tendon deterioration [74]. Although corrosion is commonly described in terms of initiation and propagation phases, the real challenge lies in how these phases are represented in service-life prediction models.

The electrochemical reactions governing corrosion can only occur if the necessary thermodynamic conditions are satisfied. In particular, the Gibbs free energy (or available electrochemical potential) must be sufficient. As shown in [75], active and long-term corrosion typically begins once the surrounding pH and alkalinity decrease, even in the presence of chlorides. Factors such as pH, alkalinity, and other chemical properties strongly influence the critical chloride threshold (C_{cr}), i.e. the concentration required to disrupt the passive film and initiate corrosion, for both reinforcing steel and prestressing wires, as recognised in [76], [77], [78]. Thus, in chloride-rich environments, the time to active corrosion (i.e., the period required to reach a stage at which long-term corrosion is triggered [79]) depends on two processes acting simultaneously: inward chloride diffusion and progressive alkalinity loss, which reduces pH buffering capacity [79]. The interplay between alkalinity loss and chloride accumulation can lead to generalised corrosion, localised pitting, or a combination of both.

In RC bridges, initiation is typically modelled through chloride ingress or carbonation until reinforcement depassivation occurs [80], [81], [82]. While this approach is widely adopted, it often neglects the combined influence of multiple processes and the variability of concrete quality.

For PT bridges, initiation modelling is more complex. Studies have distinguished between degradation initiated through intact ducts and grout [53], [64] and accelerated initiation through defective injections providing direct access to voids [56], [58]. This makes the localisation and quantification of defective injections a critical task. However, inspection of internal ducts remains difficult, data on defect distribution are scarce, and probabilistic models addressing spatial variability are still limited.

Regarding the propagation phase, models typically assume either uniform section reduction (generalised corrosion) or maximum pit depth (localised corrosion). In this context, Darmawan and Stewart [83] proposed a pit-depth evolution model specifically designed to estimate pit depth in prestressing wires. Moreover, based on experimental findings reported by [84], [85], pitting corrosion in prestressing wires may develop according to three distinct geometrical shapes: hemispherical, concave, and planar; although [85] demonstrated that the concave morphology occurs only at advanced corrosion stages and in relatively rare cases, whereas hemispherical and planar pits are observed with comparable probability. These different corrosion morphologies result in varying residual cross-sectional areas for the same pit depth; therefore, this effect should be explicitly considered when estimating the residual cross-section and the remaining capacity of corroded wires.

Pitting corrosion also affects the mechanical behaviour of prestressing wires by reducing their ultimate strain and tensile strength, and, in severe cases, their yield stress, owing to stress concentrations that develop at the base of pits. For instance, Franceschini et al. [86], proposed an experimentally validated model (the SCPS model) to estimate the reduced mechanical properties of corroded strands, as a function of the ratio of pit depth to the initial wire radius. Similarly, Yu et al. [87] developed a model to estimate the reduced mechanical properties of strands and wires as a function of the residual cross-sectional area, also supported by experimental validation.

Overall, the literature confirms that deterministic approaches are insufficient. Probabilistic formulations are essential for capturing uncertainties in initiation, propagation, and spatial variability. When sufficient data are available, probabilistic models can be tailored to specific structures and updated, provided that the temporal characteristics of the degradation process are preserved, thereby improving the relevance of reliability assessments and service-life predictions.

However, targeted data-driven models for PT bridges remain largely underdeveloped, and moving from component to system-level perspectives is critical to capture correlations among different bridge elements and their simultaneous degradation, facilitating a global assessment of these structures.

2.3.2 Mechanical Modelling Strategies for System Reliability Analysis of Multi-Girder Concrete Bridge Decks

The system reliability assessment of reinforced and prestressed concrete multi-girder bridge decks requires an appropriate representation of the mechanical interaction among longitudinal girders and transverse elements. In particular, the modelling approach should capture, to a sufficient degree of accuracy, the load distribution mechanism, the potential redistribution of internal forces after local yielding, and the influence of deterioration processes on system behaviour.

In the literature, several modelling strategies have been proposed, e.g., nonlinear finite element (NLFE) grillage models with explicit plastic redistribution, and simplified analytical grillage models embedded within probabilistic frameworks. Choosing a modelling approach depends on the objective of the analysis. Indeed, while NLFE approaches aim at reproducing the structural response up to collapse with high mechanical fidelity, time-variant probabilistic system reliability requires repeated evaluations of the structural model over a large number of stochastic realisations and time steps. Therefore, mechanical realism and computational demand need to be balanced.

In NLFE grillage models, the bridge deck is idealised as an ensemble of longitudinal and transverse beam elements, whose material and section nonlinearity are described using fibre-section formulations or plastic hinge models. These approaches explicitly simulate cracking, yielding, plastic hinge formation, and progressive redistribution of internal forces. In this regard, Conti et al. [88] employed a NLFE grillage model to evaluate the residual capacity of a reinforced concrete grillage deck affected by corrosion. The redistribution of internal forces and deformation patterns were explicitly captured, highlighting the importance of torsion–bending interaction and progressive stiffness degradation. Corrosion was localised on a 3 m long segment across the midspan section of one of the external beams of the studied deck. The Authors' analysis showed that, as corrosion progresses over time, the ductility of the external girder decreases, resulting in a reduced capacity for internal force redistribution among the remaining girders. A transition from a ductile global collapse mechanism (characterised by significant internal force redistribution) to a more brittle one (characterised by limited redistribution) was observed as corrosion develops. While providing valuable insights into the global behaviour of reinforced concrete grillage decks subjected to corrosion, the study considers only a single corrosion location and a specific governing failure mode, thereby limiting the exploration of alternative deterioration scenarios and load configurations.

In [89], Wang et al. developed a time-variant system reliability framework for deteriorated multi-girder bridges based on incremental nonlinear finite element analysis with 3D fibre beam elements. In particular, the authors distinguished among: single-girder failure mode (SGFM), multi-girder failure under elastic distribution (MGFM-ED), and multi-girder failure with plastic redistribution (MGFM-PR). Quantitative indicators describing effective load distribution and plastic redistribution capacity were introduced, and their model incorporated corrosion-induced material degradation and time-dependent ductility reduction. The nonlinear model was also calibrated using results obtained through a full-scale failure test described in [90]. However, in this case, corrosion spatial variability is disregarded, and it is assumed developing at the worst location for the analysed failure mode, i.e., at the midspan for bending failure.

Another approach to the problem is to define and quantify decks redundancy and robustness based on component and system reliability evaluations. Redundancy is quantified as the ratio between system ultimate capacity and first-member capacity, while robustness measures the residual capacity after local damage.

Within this context, for instance, [91] proposed reliability-based system factors and formal definitions of redundancy and robustness in bridge systems, obtaining system factors for different decks configurations and bending or shear failure modes. These factors were estimated based on several approximated reliability analyses accounting for nonlinear behaviour of the studied decks assuming that an ultimate system target reliability of 4.35 is achieved in the design phase of each bridge.

Sarmiento et al. [92], [93] analyse redundancy and robustness indicators defined through deterministic measures, reliability-based measures, and risk-based measures for a prestressed grillage bridge deck under two damage scenarios: overloading and chloride-induced degradation. In particular, a NLFE model is developed, and the prestressing tendons and steel reinforcement are progressively removed due to the damage caused either by overloading or corrosion. In order to reduce the computational demand, a surrogate model is developed using the Kriging metamodel methodology. Their results demonstrate that, while the system reliability of the structure can be substantially higher than the reliability of an individual element, this gap decreases as damage develops, due to the reduced ability of the deteriorated structure to redistribute internal forces. The insights provided are valuable; however, the analysis is limited to the bending failure mode and damage localised at midspan.

To reduce the computational effort required to perform system reliability analyses coupled with NLFE models, Fiorillo and Ghosn [94] combined parallel Monte Carlo simulations with NLFE with lumped plasticity of grillage decks, leading to the development of an open source software called OpenBRAIN. The model predictions were validated against results from a full-scale destructive laboratory test. In later work [95], the same authors used their OpenBRAIN software to quantify the reduction in ultimate system capacity under local damage scenarios. The Authors provided a statistical characterisation of load factors of a single girder component, of the whole deck, and in the case of damage of a component. Moreover, insights regarding the effect of different damage positions in respect to load positions are provided, highlighting that the effect of the damage is more pronounced when in proximity of an applied load and that a damage of the main girders produces a significant decrease in capacity while a slab damage has a limited impact. Despite recognising the importance of the relative position of loads and damage, the analysis considers only a single fixed load configuration.

These studies demonstrated that redistribution significantly affects system-level reliability while damage location and extent strongly influence ultimate capacity.

However, within these approaches, system reliability is evaluated under fixed structural states and fixed loads conditions rather than within a fully space and time-dependent deterioration context.

Indeed, while the NLFE grillage and fibre-beam models are demonstrated fully able in representing actual collapse of concrete grillage decks, the need of incremental-iterative equilibrium procedures becomes a limitation in a probabilistic framework where repeated structural analyses for each stochastic realisation are required. When embedded within time-variant reliability analysis, especially in the case of spatially correlated deterioration and non-fixed position traffic loads, the computational demand increases. Therefore, for reliability problems requiring millions of structural evaluations over multiple time steps, direct NLFE model becomes computationally unfeasible. The parallel Monte Carlo simulations proposed in [94] partially overcome this shortcoming. However, even within that approach, computational challenges remain when accounting for time and spatial dependencies of degradation, moving traffic loads, and multiple failure modes.

Finally, surrogate models of the grillage deck could be adopted, but for each investigate load position, analysed section and failure mode an *ad-hoc* surrogate model should be trained. Such models should then be utilised in the evaluation of the system Limit State Function (LSF), again leading to an explosion of the computational demand.

Therefore, simplified analytical grillage approaches are needed to perform system reliability analyses accounting for spatial and time-dependent degradation. For instance, Nettis et al [96] proposed an automated fragility framework for prestressed concrete girder bridges under traffic loads, aiming for large-scale fragility and portfolio-level reliability assessment. Their approach employs elastic transverse load distribution based on Courbon method accounting for geometric uncertainties and allows the estimation of the sectional capacity over time including corrosion degradation instead of pre-determined damage scenarios. As limitations, redistribution beyond the elastic stage is disregarded and the system behaviour is only accounted for through the transversal load redistribution coefficients. However, the study was able to provide the basis for the development of fragility curves of existing concrete decks accounting for the effect of degradation.

2.3.3 Adopted System Modelling of the Multi-girder concrete decks

As discussed, the literature indicates that multi-girder concrete decks exhibit partial ductility which reduces over time due to corrosion. NLFE grillage models are capable of representing the actual mechanical behaviour but cannot yet be employed when investigating the effects of corrosion accounting for both spatial and time dependent uncertainties.

The objective of the thesis is the evaluation of time-variant system reliability under spatially correlated deterioration accounting for acquired inspection data. In particular, spatial variability, correlation between component resistances, Bayesian updating using inspection data, and over time correlation of the LSFs must be modelled and kept within the whole timespan of the analysis. Therefore, the structural model must be evaluated repeatedly over many stochastic realisations, remain computationally tractable within life-cycle reliability analysis, and facilitate integration of Bayesian updating and deterioration modelling.

The modelling strategy adopted in this thesis aims to retain the essential features governing system capacity and load redistribution while ensuring analytical tractability and computational efficiency for time-dependent reliability assessment. A Daniels system mechanical model is then adopted in this thesis. In particular, for the multi girder decks the main mechanical assumptions are that the longitudinal girders act as primary load-carrying components while transverse slab and crossbeams provide load redistribution. Therefore, the Daniels System representation does not attempt to replicate the full nonlinear response of the bridge, but it provides a mechanical model abstraction of the ultimate system capacity due to the interaction of longitudinal components. Similarly, for shear-governed deterioration scenarios, the deck system is represented as a brittle Daniels System. This modelling choice is motivated by the mechanical characteristics of shear failure in reinforced and prestressed concrete members, which generally exhibit substantially lower deformation capacity and redistribution potential than flexural failure. In corroded girders, this behaviour is even more pronounced, since corrosion reduces the overall section ductility, and at the system level, a local shear failure provides limited opportunity for progressive redistribution among adjacent girders. Therefore, the brittle Daniels System formulation is adopted.

While not modelling nonlinearity, the Daniels system implicitly accounts for redistribution through the load-sharing rule. The abstraction represents a mechanically consistent reduced model suitable for spatial and time dependent reliability evaluation, and the adopted system configuration can be interpreted as a conservative approximation of the actual system load capacity accounting for redistribution.

Acknowledging the employed system modelling limitations, a NLFE model is developed for each case study presented in Chapter 8 to estimate the ultimate capacity of the decks, and to estimate the level of approximation introduced using the Daniels system modelling. The FE models are employed to estimate the system capacity accounting for load redistribution in different load cases and at different corrosion stages. The relative error of the system capacity estimated through the NLFE and the proposed Daniels system model is also computed, providing an estimation of the conservativeness of the results.

2.4 Reliability Requirements for Structures

Since the dawn of civilisation, people have placed implicit trust in the safety of structures. Today, this expectation is reflected in the extremely low failure probabilities prescribed for both new and existing structures. Two main principles guide the definition of these reliability requirements: the *As Low As Reasonably Practicable* (ALARP) principle and the *Marginal Life-Saving Costs* (MLSC) principle [11].

In the ALARP framework, structural requirements, expressed as maximum annual failure probabilities, are defined to ensure a tolerable (acceptable) risk of failure. The MLSC principle, on the other hand, seeks to align economically optimal and feasible decisions with society's preferences regarding life safety. Specifically, MLSC establishes that the cost of saving one additional statistical life must be balanced against society's willingness to pay for that risk reduction. Although the two principles differ in formulation, they converge when defining the acceptable risk region based on societal willingness to pay.

As a consequence, annual structural reliability requirements are closely intertwined with a country or region's economic development (e.g., the European Union). Parameters such as gross domestic product, life expectancy, and the balance between working and leisure time influence the target level of structural safety. In practice, this means that developing countries generally accept higher structural risks than developed ones. This difference becomes even more pronounced when discount rates are applied to structural investments, since the benefits of lives saved accrue over the service life of a structure. Higher discount rates, typical of developing countries, therefore, correspond to lower acceptable safety levels.

Moreover, economic optimisation principles can also be applied to define target failure probabilities [97]. In this case, requirements are formulated as a balance between the costs of risk-reduction measures and the potential consequences of failure. For instance, different target values may be specified for partial failures (with limited consequences) and global failures (with potentially higher direct and indirect consequences).

Table 2-1 summarises the target requirements in terms of the target reliability index, β_t , defined as: $\beta_t = \Phi^{-1}(1 - P_{ft})$. Here, Φ^{-1} denotes the inverse cumulative distribution function of the Standard Gaussian Distribution, and P_{ft} is the target failure probability determined according to the ALARP, MLSC, or economic optimisation principles.

Given the relative costs, the β_t values associated with Class 2 consequences generally satisfy life-safety requirements in developed countries, reinforcing the principle that structural safety should ultimately be constrained by economic considerations rather than by human-safety arguments [12].

Table 2-1: Target reliability index values depending on the costs of safety measures and the consequences of failure [97]

Relative costs of safety measure	Consequence of failure		
	Class 2	Class 3	Class 4
Large	3,1	3,3	3,7
Medium	3,7	4,2	4,4
Small	4,2	4,4	4,7

2.4.1 Requirements for Existing structures

The same principles can be applied when assessing either individual elements or an existing structure as a whole. However, when all other parameters are held constant, interventions on existing structures are typically associated with higher costs, which in turn lead to lower target reliability levels being deemed acceptable. Nevertheless, human-safety requirements must always be satisfied. Figure 2-2, adapted from [11] and [12], illustrates this concept by showing how the target reliability associated with the optimal cost decreases as the marginal cost of safety improvement shifts from new structures to existing ones.

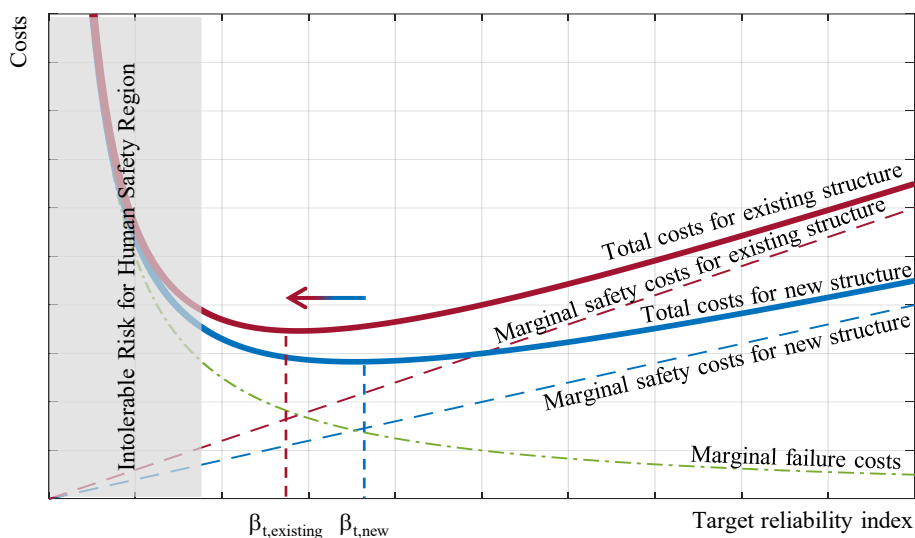


Figure 2-2: Target reliability as an economic optimisation problem constrained by the human-safety criterion.

The figure highlights a key implication: the acceptable failure probability for an existing structure is higher than that for a newly designed one. This reflects the greater efficiency of adopting conservative safety margins at the design stage, when modifications are relatively inexpensive, compared to the assessment stage of

ageing structures, when interventions are considerably more costly. Consequently, assessments of existing structures should target reliability levels that balance safety with economic feasibility, ensuring that limited resources are allocated efficiently without compromising essential life-safety requirements.

2.5 Research gaps and summary of the Chapter

Based on the state-of-the-research analysis, significant gaps can be identified in estimating the remaining service life of concrete bridges: (i) reliability analyses often focus on individual components rather than adopting a structural system perspective; (ii) the interaction of multiple failure modes at the component level, together with their correlations, is often overlooked; (iii) modelling of defect occurrence and distribution within injected ducts of PT bridges remains limited, with few probabilistic models explicitly addressing spatial variability; (iv) existing methodologies for collecting data on the conservation status of PT bridges have limited applicability for supporting the development of probabilistic models of defect occurrence and distribution; (v) corrosion initiation is typically modelled without considering the quality of the surrounding concrete; (vi) calibration and updating of probabilistic degradation models are often performed without preserving the temporal characteristics of the underlying processes.

These gaps are decisive. They show that while probabilistic analyses are increasingly recognised as a valuable tool, their application have remained fragmented, either confined to component-level analyses or based on overly simplified degradation assumptions. Moreover, PT bridge deterioration has not been systematically studied, integrating defect occurrence, spatial variability, and concrete quality into a unified framework.

These shortcomings highlight the need to further develop existing methods for service-life assessment of concrete bridges. Current approaches either remain limited to component-level evaluations, neglecting system interactions, or adopt simplified degradation assumptions that cannot capture the spatial and temporal variability observed in practice. Moreover, the lack of probabilistic models for tendon defects and the limited integration of inspection data restrict their ability to support decision-making. Managers must prioritise interventions across hundreds of ageing structures under budget constraints, and in this context, deterministic or localised assessments may yield either overly conservative predictions, leading to premature interventions and wasted resources, or under conservative outcomes that compromise safety.

3 Objectives, Research Process, and methods

The assessment and management of existing concrete bridges emerge from the state-of-the-research as a pressing and complex challenge that requires immediate attention. The growing age of bridge inventories worldwide calls for systematic evaluation of their current safety levels and the estimation of the remaining service life during which these structures can be considered reliable and serviceable. Beyond assessing current conditions, it is essential to establish rational strategies for planning inspections, tests, and monitoring campaigns, progressively building a cumulative knowledge base required to improve the accuracy and relevance of each subsequent assessment. Given the inherent uncertainties in material properties [98], environmental actions, degradation processes, and model assumptions, probabilistic approaches represent the most suitable tools to ensure that assessments account for these uncertainties quantitatively [33]. In this context, the objective of this thesis is to employ reliability analyses as a rational basis for bridge management, enabling the prioritisation of interventions and the optimisation of available resources.

Building on the research gaps identified in Chapter 2, the thesis addresses four main RQs formulated in Chapter 1 through two main objectives: (i) to develop a reliability framework for estimating the service life of concrete bridges that explicitly accounts for degradation processes in both time and space. The framework is designed to enable rational service-life estimation by considering interactions among structural elements and their associated failure modes, including spatial and temporal dependencies; (ii) to integrate inspection and monitoring data into the reliability framework in order to enhance the relevance of service-life estimates and, where possible, extend the bridge operational lifespan. This objective aims to achieve more relevant, case-specific estimates by reducing uncertainty, thereby supporting more efficient infrastructure management and sustainable decision-making.

These objectives are pursued through a combination of probabilistic modelling techniques and in situ inspection and testing campaigns. A five-step research process is defined and described in the following:

- (i) **Literature review:** the first step is a review of literature and guidelines concerning the inspection of PT bridges, aiming at developing a methodology to detect, localise, and quantify defects within PT ducts.

- In parallel, the state of the art in reliability analysis of concrete structures subjected to degradation is examined.
- (ii) **Time-dependent System Reliability Framework:** building on the existing literature, an approach is developed to calculate the time-dependent system reliability of a concrete deck exposed to degradation and multiple failure modes, also identifying the main variables influencing reliability estimation. Empirical evidence on corrosion development from published studies of existing structures is incorporated into the analysis.
 - (iii) **Inspection of PT bridges:** the inspection methodology for PT bridges is then extensively applied by Italian infrastructure management companies, enabling the collection of data on the actual conservation status of PT girders.
 - (iv) **Probabilistic model of injection defects in PT girders:** based on statistical and probabilistic analysis of the inspection data, a probabilistic model for the occurrence and localisation of injection defects and corrosion spots is derived and thus used in the estimation of the spatial and time-dependent reliability of PT girders and decks.
 - (v) **Updating of the Service-Life estimation using data:** the reliability framework is integrated with updating methods that preserve the temporal characteristics of corrosion processes, allowing for refined estimations of bridge service life.

This research process is directly connected with the research questions, and several methods are utilised to conduct the research, as detailed below:

- RQ1: investigate system configurations that enable the integration of spatial and temporal dependencies, and different post-failure consequences by:
 - m.1 employing logical system modelling concepts to represent the interaction among different structural components;
 - m.2 defining correlation structures between random variables through copulas, and modelling spatially dependent variables through random fields;
 - m.3 using parametric stochastic processes to describe corrosion progression and evaluate its influence on structural capacity over time;
 - m.4 defining system typologies based on failure consequences and associating them with appropriate target reliability levels;
 - m.5 incorporating survival information into system reliability analyses to improve reliability estimation and extend predicted service-life when supported by empirical evidence.

- RQ2: account for the interaction among different corrosion mechanisms and their dependence on concrete quality and conservation state by:
 - m.1 recognising that corrosion activation conditions vary according to the quality of the surrounding concrete;
 - m.2 using logical system modelling to capture these differences and to represent triggering of corrosion processes under varying material and environmental conditions.
- RQ3: enhance the reliability calculation of PT bridges by developing probabilistic models of tendon degradation informed through inspection data, by:
 - m.1 performing statistical analyses of inspection results to quantify variability in tendon condition and to identify correlations between voids and corrosion occurrence;
 - m.2 defining probabilistic models for the number and spatial distribution of voids and corrosion spots, and characterising their joint dependence;
 - m.3 proposing an advanced simulation procedure that integrates these probabilistic models into the spatial and time-dependent system reliability framework;
 - m.4 conducting sensitivity analyses to determine the impact of new degradation-related variables on the reliability index and to inform future inspection planning.
- RQ4: integrate multi-source data from inspection, testing, and monitoring within the system reliability framework to improve the relevance of service-life estimation for RC and PT bridges, by:
 - m.1 developing Bayesian updating procedures that preserve the temporal characteristics of corrosion processes;
 - m.2 combining directly measurable data (e.g., material strength) and indirect evidence (e.g., chloride ingress, carbonation tests, survival information) to perform posterior reliability analyses;
 - m.3 evaluating the potential service-life extension achievable through the incorporation of empirical data.

The applicability of the developed approaches is studied through representative case studies. The proposed methods are applied to both a post-tensioned and a reinforced concrete bridge deck. In the first case study, the influence of defect distribution and the time evolution of system reliability are assessed, along with the spatial distribution of failed sections. In the second case study, monitoring and testing data are integrated into the analysis using different Bayesian updating techniques to assess their impact on the reliability assessment.

Finally, these objectives and methods are consistently addressed across the thesis chapters. Chapter 4 introduces a comprehensive system reliability framework that combines nested structural modelling, time-dependent degradation processes, and Bayesian updating procedures. This foundation is progressively expanded in the subsequent chapters through the development of data-driven probabilistic models for tendon defects (Chapter 5), their implementation within a spatially dependent system reliability formulation (Chapter 6), the modelling of corrosion initiation and propagation under varying concrete qualities and conservation states (Chapter 7), the application to two case studies representing a PT and an RC deck (Chapter 8).

4 Framework for System Reliability Analysis and Updating

In response to the RQs presented in Section 1.2, a comprehensive framework is developed to perform system reliability analyses encompassing multiple failure modes, spatial and time-dependent degradation, and model updating through the integration of multi-source data.

In particular, different logical system configurations are employed to address RQ1, explicitly considering both spatial correlations [30] and temporal dependencies [37], thereby leading to a spatially dependent time-variant system reliability analysis. Moreover, sensitivity analyses [40] are conducted to identify the dominant variables at different points in time, corresponding to distinct degradation levels, thereby guiding inspection and monitoring strategies.

Parametric stochastic processes are used to model corrosion-induced degradation at the component level for both RC and PT bridges. The influence of injection voids, along with their occurrence and spatial distribution, on the degradation of PT tendons and the overall system reliability of PT decks is discussed in Chapters 5 and 6 (addressing RQ3). The stochastic input parameters for the corrosion models are defined for each degrading component and include a correlation model based on components inter- distance, thereby capturing spatial variability. The evaluation of time-dependent capacity for each component allows the direct incorporation of the corrosion process into the framework. Moreover, by appropriately modelling the interactions among input parameters under different concrete qualities and conservation states, the framework can represent the variability in degradation mechanisms (see Section 7), thus addressing RQ2.

Regarding RQ4, the relevance of system reliability calculations is enhanced by integrating information obtained from inspections and monitoring [42], [45], [47]. Therefore, procedures for incorporating new information have been embedded within the framework. This integration enables the progressive updating of probabilistic models as new data becomes available.

The framework provides insights into failure behaviour across multiple levels, from individual failure modes and sections to elements and the entire structural system. Consequently, the spatial distribution of failing sections can be analysed for each failure mode, providing valuable information for the design and optimisation of

inspection and testing campaigns over time. Additionally, different spatial modelling techniques and assumptions can be applied to evaluate their influence on system reliability outcomes.

The ultimate goal of the framework is to advance the estimation of existing structural performance by addressing current research gaps and delivering ready-to-use analytical tools for the efficient management of ageing infrastructure assets.

Specifically, the framework is tailored to existing concrete bridge decks, as illustrated in Figure 4-1, which depicts a typical PT deck composed of parallel girders. In this example, bending and shear are shown as representative failure modes, along with their spatial locations, to demonstrate the multi-level and spatially explicit nature of the proposed modelling approach.

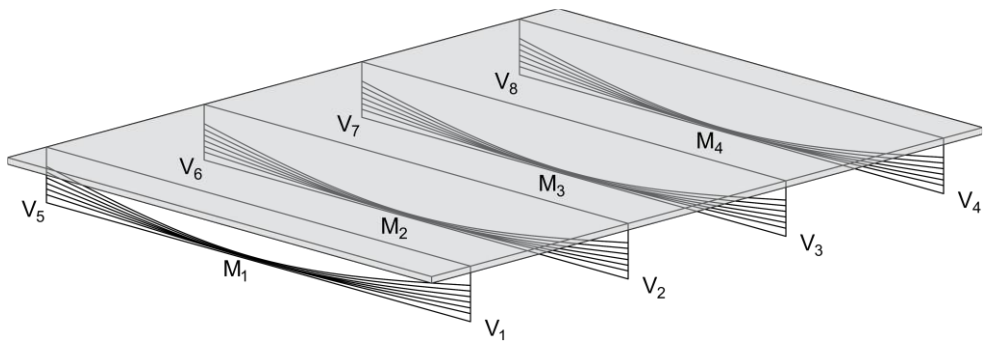


Figure 4-1: Typical PT deck composed of parallel girders

The overall framework is summarised in Figure 4-2 and is described in detail in the following Sections of Chapter 4. To preserve temporal dependencies of variables within the simulation procedures, they are grouped into three categories: (1) *Time-independent variables* remain fixed throughout the

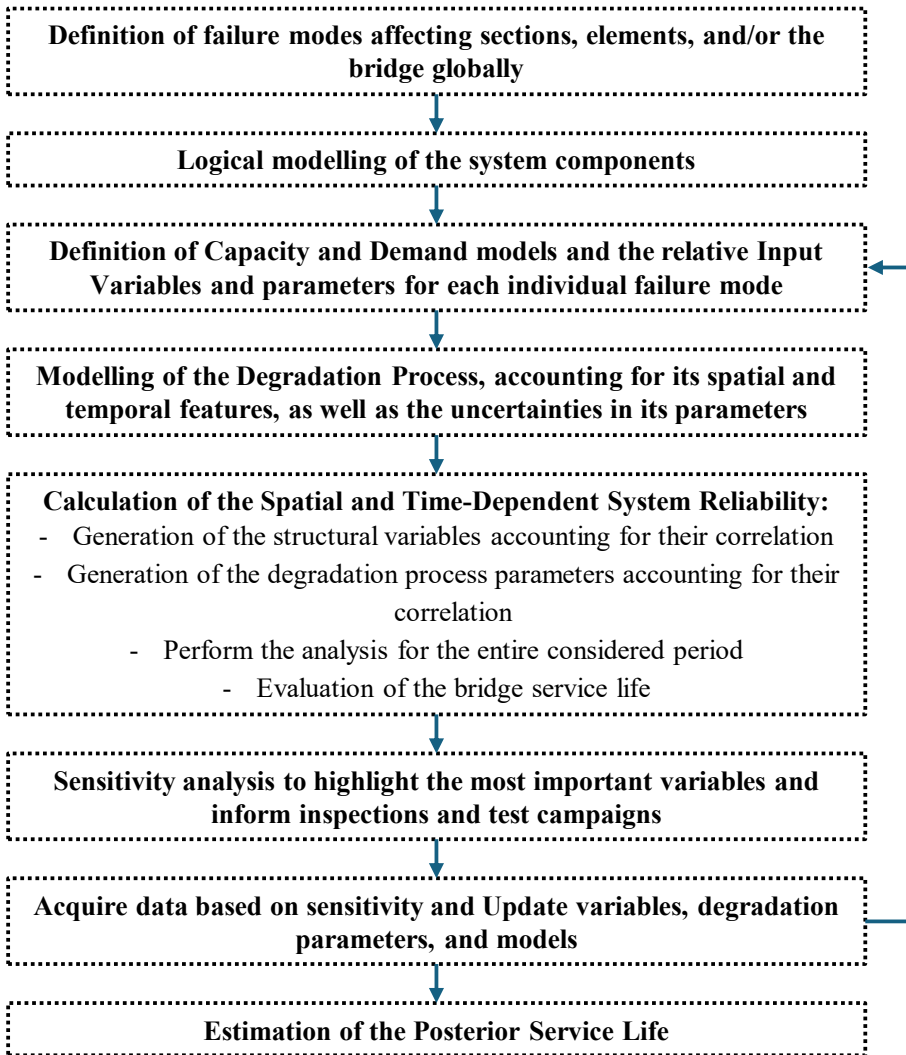


Figure 4-2: System Reliability Framework

analysis. Examples include concrete strength and steel yield strength. They are sampled once at the beginning of the simulation, and their realisations are considered fully correlated over time. (2) *Re-sampled variables* are updated at each time step and treated as independent between time steps. This category includes variable loads, represented by their annual maximum distribution, as well as temperature, corrosion current density, corrosion rate, and time of wetness. Although they are modelled as time-independent, their realisations are assumed to be uncorrelated across different time steps. (3) *Corrosion-related variables* follow parameterised stochastic processes. Examples are the time to corrosion initiation

and corrosion pit depth, which evolve according to degradation models. Their governing parameters (e.g., apparent surface chloride concentration, relative humidity) can be interpreted as hyperparameters within the hierarchical reliability model. These are sampled only at the start of the simulation, while the chosen degradation process model determines the subsequent evolution of the variables.

4.1 Structural system reliability

System reliability refers to the combined reliability of all structural elements that form the bridge system. In concrete decks, multiple failure modes may occur in each girder and at multiple sections along its span. These must therefore be incorporated into the system definition. Accordingly, the bridge is modelled as a nested system: the deck is treated as a system composed of girders; each girder is treated as a system of its sections; and each section is treated as a system of its potential failure modes. Spatial correlations among all these components are explicitly considered.

In this context, failure is defined as the occurrence of at least one of the individual failure events, representing the union of all considered failure modes. This leads to the following LSF [36]:

$$g_G(\mathbf{X}, \mathbf{S}) = \min_f \left[\min_i \left(g_{f,i}(\mathbf{X}, \mathbf{S}) \right) \right] \quad (1)$$

where $g_G(\mathbf{X}, \mathbf{S})$ is the LSF for a girder, $g_{f,i}(\mathbf{X}, \mathbf{S})$ is the LSF for the i -th section and the f -th failure mode, and $\mathbf{X}$ and $\mathbf{S}$ denote the vectors of the Capacity- and Demand-related input variables, respectively.

The LSF for the deck, modelled as a series system of girders, is then given by:

$$g_S(\mathbf{X}, \mathbf{S}) = \min_j \left(g_{G,j}(\mathbf{X}, \mathbf{S}) \right) \quad (2)$$

in which $g_S(\mathbf{X}, \mathbf{S})$ represents the deck LSF and $g_{G,j}(\mathbf{X}, \mathbf{S})$ is the LSF for the j -th girder calculated accordingly to Eq. (1).

Alternatively, when multiple components jointly contribute to the deck capacity, the system can be represented as a series arrangement of multiple brittle Daniels Systems (DSs) [22]. In this case, the LSF is defined as:

$$g_{bDS}(\mathbf{X}, \mathbf{S}) = \min_f \left\{ \min_i \left[\max_j \left[(n - j + 1) C_{f,j,i}(\mathbf{X}) \right] - \sum_j D_{f,j,i}(\mathbf{S}) \right] \right\} \quad (3)$$

In Eq. (3) $g_{bDS}(\mathbf{X}, \mathbf{S})$ is the LSF, $C_{f,j,i}(\mathbf{X})$ is the capacity and $D_{f,j,i}(\mathbf{S})$ is the demand corresponding to f -th failure mode of the j -th girder evaluated at section i , while the number of sections in parallel is denoted by n . To properly capture load redistribution after girder failures, the girder capacities at each section i are arranged

in ascending order. The set of sections includes all those analysed under each failure mode scenario.

As discussed in Sections 2.3.2 and 2.3.3, although Eq. (3) provides an analytically closed-form representation of system capacity, its applicability and level of approximation must be verified against a NLFE grillage model capable of more accurately representing the structural behaviour of the deck under investigation. Indeed, Eq. (3) is particularly advantageous because it preserves the spatial and temporal correlation of degradation and allows the consideration of multiple load positions within the probabilistic framework. However, it does not explicitly model post-elastic redistribution. Therefore, the use of an NLFE model allows, for selected load configurations and degradation states, the quantification of the approximation error associated with the DS representation. Moreover, the capacity of crossbeams and slab elements to redistribute loads transversely must be verified, as the validity of Eq. (3) relies on the assumption that transverse load transfer remains effective.

4.2 Limit state functions for system reliability

The general failure event associated with the f -th failure mode in the i -th section ($F_{f,i}(\mathbf{X}, \mathbf{S})$) is defined through the following equation:

$$F_{f,i}(\mathbf{X}, \mathbf{S}) : C_{f,i}(\mathbf{X}) - D_{f,i}(\mathbf{S}) < 0 \quad (4)$$

where $C_{f,i}(\mathbf{X})$ and $D_{f,i}(\mathbf{S})$ are the capacity and the demand associated with the f -th failure mode in the i -th section, respectively.

The failure probability corresponding to the event $F_{f,i}(\mathbf{X}, \mathbf{S})$ is evaluated using the associated LSF: $g_{f,i}(\mathbf{X}, \mathbf{S}) = C_{f,i}(\mathbf{X}) - D_{f,i}(\mathbf{S})$ and computing the probability that $g_{f,i}(\mathbf{X}, \mathbf{S}) < 0$. Such LSFs must be defined for all relevant failure modes, which then serve as inputs for the system formulations presented in Eqs. (1) to (3).

Model uncertainties can also be incorporated when defining $C_{f,i}(\mathbf{X})$ and $D_{f,i}(\mathbf{S})$. Following the provision by the JCSS model code, [99], the LSF $g_{f,i}(\mathbf{X}, \mathbf{S})$ can be modified to accommodate model uncertainties as:

$$g_{f,i}(\mathbf{X}, \mathbf{S}) = \theta_{f,C} C_{f,i}(\mathbf{X}) - \theta_{f,D} D_{f,i}(\mathbf{S}) \quad (5)$$

where $\theta_{f,C}$ and $\theta_{f,D}$ are the capacity and the demand model uncertainties, respectively. This modified LSF is then applied within Eq. (1) for each section and failure mode considered in the system reliability analysis.

4.3 Time-dependent structural reliability

As described in Section 2.2.1, an effective way to address a time-dependent reliability problem is to discretise time and reformulate it as a sequence of time-independent problems. Accordingly, the failure event defined in Eq. (4) and the relative LSF (Eq. (5)) can be redefined as point-in-time failure events extended to include time-dependency and model uncertainties:

$$F_{f,i}(\mathbf{X}, \mathbf{S}, t) : \theta_{f,C} C_{f,i}(\mathbf{X}, t) - \theta_{f,D} D_{f,i,\max(t)}(\mathbf{S}) < 0 \quad (6)$$

$$g_{f,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{f,C} C_{f,i}(\mathbf{X}, t) - \theta_{f,D} D_{f,i,\max(t)}(\mathbf{S}) \quad (7)$$

where t is the discretised time, $C_{f,i}(\mathbf{X}, t)$ is the structural capacity at time t , and $D_{f,\max(t)}$ is the corresponding maximum demand within the discretised time t . In particular, $C_{f,i}(\mathbf{X}, t)$ can be defined as follows:

$$C_{f,i}(\mathbf{X}, t) = C_{f,i}(\mathbf{X}, 0) - I[t - T_{H,i}(\boldsymbol{\eta})] H_i(\boldsymbol{\eta}, t) \quad (8)$$

where $T_{H,i}(\boldsymbol{\eta})$ is the time needed for the degradation process to become active, depending on the vector of degradation stochastic parameters $\boldsymbol{\eta}$ at section i , $I[\cdot]$ is the indicator function, assuming value 1 or 0 when the argument is higher or smaller than 0, respectively. $H_i(\boldsymbol{\eta}, t)$ represents the degradation development function at section i . The time dependence of capacity is therefore governed by the degradation processes described in Chapter 7.

The formulation (8) allows for the inclusion of spatial dependencies, since for different sections i a correlation model of the input variables and degradation parameters can be defined based on the distance between pairs of sections, for example, through random fields.

If time is discretised annually, the resulting annual reliability can be compared with the target reliability indices prescribed in standards [11] (see also Section 2.4), allowing the estimation of the structure service life. The Eqs. from (1) to (3) can be adapted accordingly.

However, as discussed in Section 2.2.1, the annual reliabilities calculated in this manner consider each year separately, ignoring valuable information such as the fact that the structure has survived up to the considered time step. This survival information may reveal critical for service-life estimation, as a structure that has already reached an advanced age without failure provides evidence of its reliability, and this knowledge should reduce the estimated probability of imminent failure.

To incorporate this aspect, the cumulative failure event $F_{f,i}(\mathbf{X}, \mathbf{S}, [0, t])$ associated with the f -th failure mode in the i -th section, is defined:

$$F_{f,i}(\mathbf{X}, \mathbf{S}, [0, t]) = \{ \exists \tau \in [0, t] : g_{f,i}(\mathbf{X}, \mathbf{S}, \tau) < 0 \} \quad (9)$$

Moreover, the cumulative survival events $S_{f,i}(\mathbf{X}, \mathbf{S}, [0, t])$ is defined as the complementary of $F_{f,i}(\mathbf{X}, \mathbf{S}, [0, t])$, and their probabilities $P(F_{f,i}(\mathbf{X}, \mathbf{S}, [0, t]))$ and $P(S_{f,i}(\mathbf{X}, \mathbf{S}, [0, t]))$, are also defined.

Similarly, the cumulative failure event for a component and for the series system or brittle DS can be defined as follows:

$$\begin{aligned} F_G(\mathbf{X}, \mathbf{S}, [0, t]) &= \{\exists \tau \in [0, t] : g_G(\mathbf{X}, \mathbf{S}, t) < 0\} \\ F_S(\mathbf{X}, \mathbf{S}, [0, t]) &= \{\exists \tau \in [0, t] : g_S(\mathbf{X}, \mathbf{S}, t) < 0\} \\ F_{bDS}(\mathbf{X}, \mathbf{S}, [0, t]) &= \{\exists \tau \in [0, t] : g_{bDS}(\mathbf{X}, \mathbf{S}, t) < 0\} \end{aligned} \quad (10)$$

These formulations, along with their relative survival events and probabilities, are used to calculate conditional failure probabilities given survival events.

Bayesian updating of the cumulative failure probability can then be performed as (notation simplified for clarity by omitting $\mathbf{X}$ and $\mathbf{S}$):

$$P(F_{f,i}([0, t])|E) = \frac{P(F_{f,i}([0, t]) \cap E)}{P(E)} \quad (11)$$

where E is the conditioning event (e.g., corrosion indication, damage indication, or proof load test [100], [101], [102]). When E coincides with the survival event, the conditional probability of failure in year t , given the survival until time t_A , can then be expressed in the four cases as:

$$\begin{aligned} P(F_{f,i}([0, t])|S_{f,i}([0, t_A])) &= \frac{P(F_{f,i}([0, t]) \cap S_{f,i}([0, t_A]))}{P(S_{f,i}([0, t_A]))} \\ P(F_G([0, t])|S_G([0, t_A])) &= \frac{P(F_G([0, t]) \cap S_G([0, t_A]))}{P(S_G([0, t_A]))} \\ P(F_S([0, t])|S_S([0, t_A])) &= \frac{P(F_S([0, t]) \cap S_S([0, t_A]))}{P(S_S([0, t_A]))} \\ P(F_{bDS}([0, t])|S_{bDS}([0, t_A])) &= \frac{P(F_{bDS}([0, t]) \cap S_{bDS}([0, t_A]))}{P(S_{bDS}([0, t_A]))} \end{aligned} \quad (12)$$

where both $P(F_{f,i}([0, t]) \cap S_{f,i}([0, t_A]))$ and $P(S_{f,i}([0, t_A]))$ can be evaluated using conventional structural reliability techniques combined with the outcrossing approach, [97]. These equations are valid for $t < t_A$, while $F_{f,i}([0, t])$ and $S_{f,i}([0, t])$ denote the four different cases of failure at section i for failure mode f , or at component, series system, or brittle DS failure.

Moreover, since target reliabilities are typically specified as annual values [11], the posterior annual point-in-time failure probability for any of the four cases can be calculated as:

$$\begin{aligned}
 P(F.(t)|S.([0, t_A])) &= \frac{P(F.(t) \cap S.([0, t_A]))}{P(S.([0, t_A]))} \\
 &= \frac{P(F.(t) \cap S.([0, t_A]))}{1 - P(F.([0, t_A]))}; t > t_A
 \end{aligned} \tag{13}$$

This conditional formulation is particularly valuable because it integrates survival information into the reliability analysis at the failure mode, component, and system levels, ensuring that service-life predictions are not overly conservative and reflect the actual observed performance of the structure.

4.4 Bayesian direct and indirect updating of structural reliability

Direct updating applies to variables in the reliability models (see Eqs. from (1) to (7)) that can be directly measured, such as the compressive strength of concrete or the yield stress of steel rebars. In this case, the parameters Θ of their prior distribution (e.g., mean, variance, scale) are treated as hyperparameters, i.e., random variables updated based on test outcomes (see also ISO 2394:2015 [11]). Given the prior distribution $f'_X(x|\Theta)$ of the variable, the likelihood function representing the n test outcomes x_r is defined as: $L(x_1, \dots, x_n|\Theta) = \prod_{r=1}^n f'_X(x_r|\Theta)$, where the values x_r are observed and the parameters in vector Θ are unknowns. The joint prior distribution $p'(\Theta)$ of the parameters is chosen based on relevant literature, known ranges of variation, or mathematical convenience (e.g., conjugate priors). The posterior distribution of the parameters is then:

$$p''(\Theta|x_1, \dots, x_n) = \frac{1}{c} p'(\Theta) L(x_1, \dots, x_n|\Theta) \tag{14}$$

where c is the normalising constant. The posterior distribution of the input variable, i.e., the predictive distribution, is then obtained using the total probability theorem to integrate out the uncertainties associated with the model parameters:

$$f''_X(x|x_1, \dots, x_n) = \int_{\Theta} f'_X(x|\Omega) p''(\Omega|x_1, \dots, x_n) d\Omega \tag{15}$$

Indirect updating applies to variables that cannot be directly measured, such as the parameters of stochastic degradation models ($M(\Theta)$). The prior distributions of these parameters are typically drawn from the literature, while the prior distribution of the model output ($f'_O(O)$) is obtained through the model function using these prior parameter distributions.

If an inverse of the model $M(\Theta)$ exists ($M^{-1}(\Theta)$), the likelihood function, $L(o_1, \dots, o_n|\Theta)$, can be constructed by fitting the n values Θ obtained from

$M^{-1}(\Theta|o_r)$ to a suitable distribution, where o_r are the observations. This approach captures both data bias and dispersion, avoiding the assumption of independent, normally distributed measurement errors, while preserving the fundamental interpretation of the likelihood as the probability of observing the data given a realisation of the parameter Θ .

When $M^{-1}(\Theta)$ does not exist, the likelihood function $L(o_1, \dots, o_n|\Theta)$ can instead be defined through stochastic calibration of the model parameters using the observations o_r .

In both cases, Eqs. (14) and (15) are utilised to compute the posterior distribution of the parameters, $p''(\Theta|o_1, \dots, o_n)$, and the predictive distribution of the model output, $f''_O(O|o_1, \dots, o_n)$.

Furthermore, when instrumentation is employed to detect flaws, cracks, or other forms of damage in structures, the Probability of Detection (PoD) curve associated with the instrument can be used to define the likelihood function for the observed detections. In particular, the PoD curve links the damage size d_s to the probability of its detection $P(D_I|d_s)$, explicitly accounting for uncertainties in signal processing, measurement technology, operational conditions, and human factors. Therefore, for each detected damage instance, a corresponding likelihood function can be defined based on the PoD curve, which represents the probability of the observation given the possible true damage sizes. For a given prior probability distribution of the damage size, $f_{d_s}'(d_s)$, the posterior distribution can be updated using Bayes' theorem consistently with Eq. (14):

$$f_{d_s}''(d_s|D_I) = \frac{1}{c} P(D_I|d_s) f_{d_s}'(d_s) \quad (16)$$

where $f_{d_s}''(d_s|D_I)$ is the posterior damage size distribution and $P(D_I|d_s)$ is the PoD intended here as the likelihood function.

This formulation allows the posterior distribution of damage size to reflect both instrument performance and prior uncertainty. The approach is particularly relevant in non destructive test (NDT) applications, where measurements are indirect and subject to detection uncertainty.

Finally, when applied loads, load effects, or element capacities are measured or monitored, the collected data can be utilised to reduce the model uncertainties associated with these quantities. In this case, indication events are typically represented as intervals or ranges, for which the corresponding probabilities can be evaluated by integrating the model-uncertainty probability density function (PDF) over the specified range. Formally, if θ_M denotes the model uncertainty variable with PDF $f_{\theta_M}(z)$, and the measured range is $[z_1, z_2]$, then the conditional probability of indication event (I) is:

$$P(I) = \int_{z_1}^{z_2} f_{\theta_M}(z) dz \quad (17)$$

Given such an indication event, a truncated model uncertainty PDF can be defined:

$$f_{\theta_M|I}(z) = \frac{f_{\theta_M}(z)}{P(I)} \text{ for } z \in [z_1, z_2] \quad (18)$$

This truncation reduces model uncertainty, thereby increasing the relevance of the reliability assessment.

In the case of equality information, where the instrumentation provides a specific measured value rather than a range, the observation can be modelled as a deterministic realisation of the uncertain variable. To maintain consistency, a measurement uncertainty variable U_{meas} may be introduced to represent the uncertainties associated with the measurement process, leading to the definition of the new variable:

$$U_{\theta} = \hat{\theta}_M \cdot U_{meas} \quad (19)$$

However, in most practical applications, measurement uncertainties are smaller than model uncertainties, thereby increasing the relevance of overall reliability calculations. By incorporating both interval-based and equality-based measurement outcomes into the Bayesian updating scheme, the reliability analysis becomes progressively more informed and reflective of actual structural conditions.

4.5 Summary of the Chapter

The developed framework accounts for multiple failure modes, explicitly incorporates spatial and time-dependent degradation, and integrates Bayesian updating techniques to incorporate empirical evidence from inspections and tests. To address the challenges posed by spatial dependencies and correlations among failure modes, structural reliability is formulated using a nested system approach, in which decks are modelled as systems of girders, girders as systems of sections, and sections as systems of failure modes, while accounting for spatial correlations among all components.

Time-dependent reliability is addressed by discretising time into annual intervals and by explicitly accounting for degradation processes in the limit state formulation. This approach allows for the evaluation of the residual life of existing structures within the framework of current standards (e.g., [11]). Particular attention is given to conditional failure probabilities, which enable the inclusion of survival information in reliability assessment through time-variant reliability approaches [37], [47], [103]. This aspect may reveal crucial for service-life assessment, as it ensures that predictions reflect the observed performance of the structure over time.

In addition, Bayesian updating procedures have been introduced for both directly and indirectly measurable variables, damage size distribution, and direct information gathering. These methods enable refining prior models by incorporating empirical data from inspection, testing, and monitoring activities into the system reliability framework.

The proposed framework addresses the RQs outlined in Section 1.2 and establishes the methodological foundation for the system reliability analysis of concrete bridges. Moreover, it provides a basis for integrating specific degradation models that capture differences in corrosion progression between PT and RC structures and across different concrete qualities, as presented in the subsequent chapters.

Specifically, Chapters 5 and 6 focus on the development of probabilistic models for assessing the spatial degradation of tendons in PT bridges and their implementation within the proposed framework. Chapter 7, instead, presents the modelling of corrosion degradation for different concrete qualities and its updating using in situ test data. These models can be directly applied to the corrosion assessment of RC bridges, while specific considerations for PT bridges are explicitly discussed in Section 7.3.

5 Data-Driven Probabilistic Modelling of Defects in Post-Tensioned Bridge Girders

RQ3 is addressed explicitly in this and the following Chapters. As widely recognised in the literature [55], [78], [104], [105], [106], [107], defective grout injections have a detrimental effect on the durability of PT bridges. These defects are considered hidden because they cannot be identified through conventional visual inspections, and their detection is therefore of primary importance. To this end, a dedicated inspection methodology is developed to detect and localise hidden defects, including voids, corrosion spots, and their simultaneous occurrence, within PT tendons. Based on the data collected from these inspections, the spatial and statistical distributions of the identified defects are analysed and modelled, and a possible relationship between the presence of voids and the onset of corrosion is investigated. At the same time, the development of probabilistic models representing the occurrence and spatial distribution of such defects within ducts is essential for service-life estimation of PT structures.

5.1 Inspection methodology

The inspection methodology is developed to provide an initial dataset for the development of predictive probabilistic models for the spatial analysis of tendon corrosion degradation. It was first tested in a laboratory setting on PT specimens (Figure 5-1), to calibrate the procedures and instrumentation, and subsequently applied in situ on real bridges (Figure 5-2 and Figure 5-3) to demonstrate its effectiveness under operational conditions. The procedure is summarised as follows:

- a) *Visual inspection.* A preliminary visual examination of the bridge is conducted to assess overall conservation condition and detect visible defects affecting structural elements or details.
- b) *Determination of the number of tendons to inspect.* The number of tendons to be examined is determined following the method proposed by the Federal

Highway Administration (FHWA) [106], adapted to the Italian bridge stock. The approach assumes that inspection outcomes follow a hypergeometric distribution.

- c) *Determination of tendons layout.* The layout of the tendons is determined using Ground Penetrating Radar (GPR). When design drawings are available, their accuracy is verified against GPR results.
- d) *Localisation and quantification of defects within the ducts.* Ultrasonic Tomography (UT) is used to detect the location of potential defects along the tendon layout. Subsequently, endoscopy is used to quantify the defects in terms of void and to assess the presence of corrosion on tendons.
- e) *Determination of actual prestressing force on PT wires.* The residual prestressing force is evaluated using X-ray diffraction techniques.

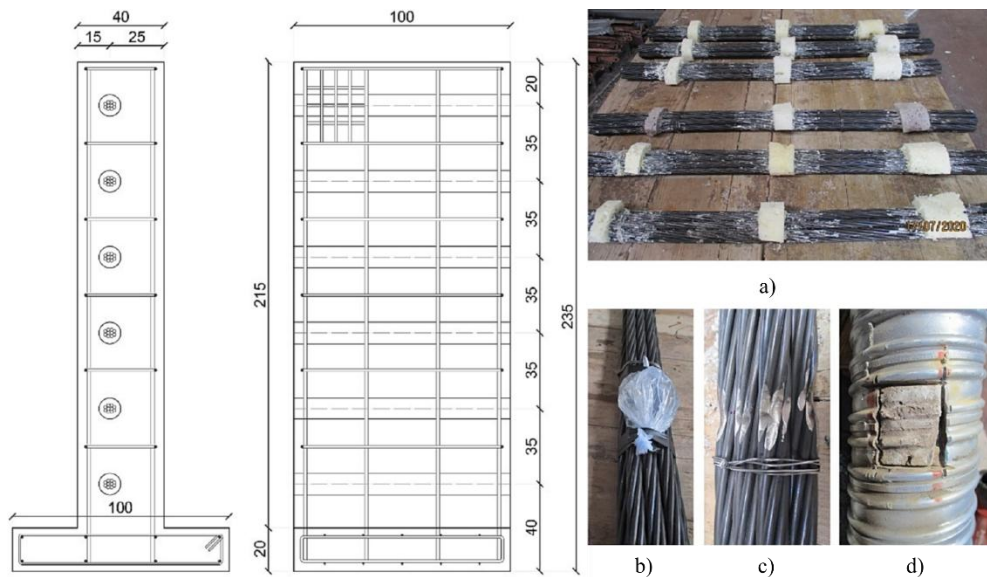


Figure 5-1: Main geometrical dimension of the specimens (left); types of damages simulated (right). a) voids, b) bleed water, c) damaged wires; d) damaged duct.



Figure 5-2: In situ execution of GPR tests.

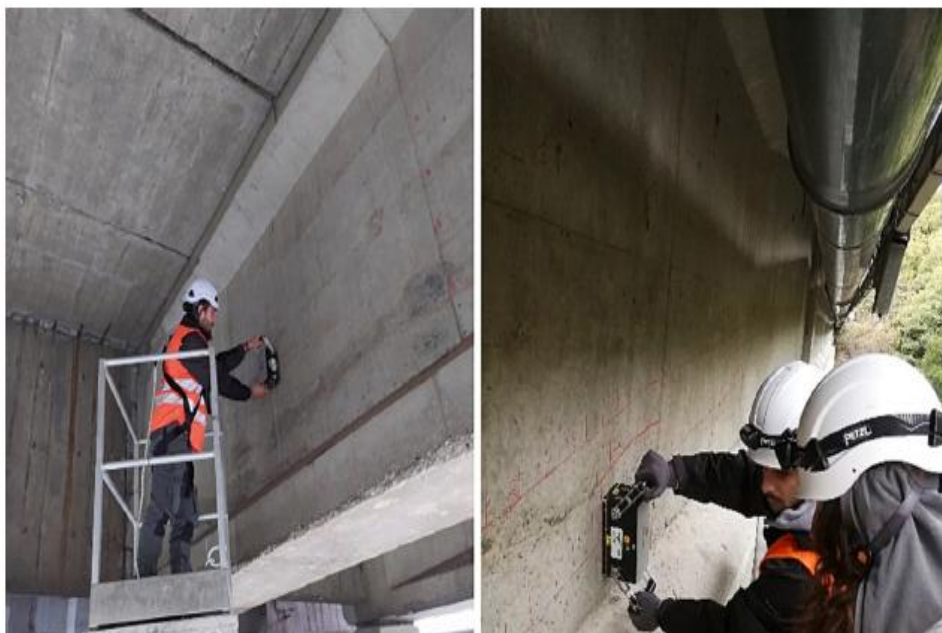


Figure 5-3: In situ execution of UT tests.

5.2 Data gathering

The developed methodology was extensively applied on PT decks in Italy to collect data, primarily through steps *c* and *d*, for the development of the probabilistic

models of defect occurrence and distribution. Among the inspected structures, 53 bridges comprising 192 girders were selected for analysis. These girders share similar design and construction methodologies, ensuring comparability of results. The selected bridges are simply supported with I-shaped girders, built during the 1960s, and span lengths ranging from 20 to 45 m (average span 35 m). Given their comparable design, construction period, and prestressing technology, the PT girders were expected to exhibit similar defect characteristics. Figure 5-4 illustrates a typical example of the analysed bridges and the cable layout within the beams.

For each girder, cables were inspected in critical zones, i.e., at beam ends, curvature changes, and mid-span regions, where defects are most likely to occur due to construction practices. After assessing tendon layouts with GPR and comparing them against design documentation (Figure 5-5), UT scans were performed at 10 cm intervals along the tendon path to identify potential voids within the ducts (Figure 5-6).

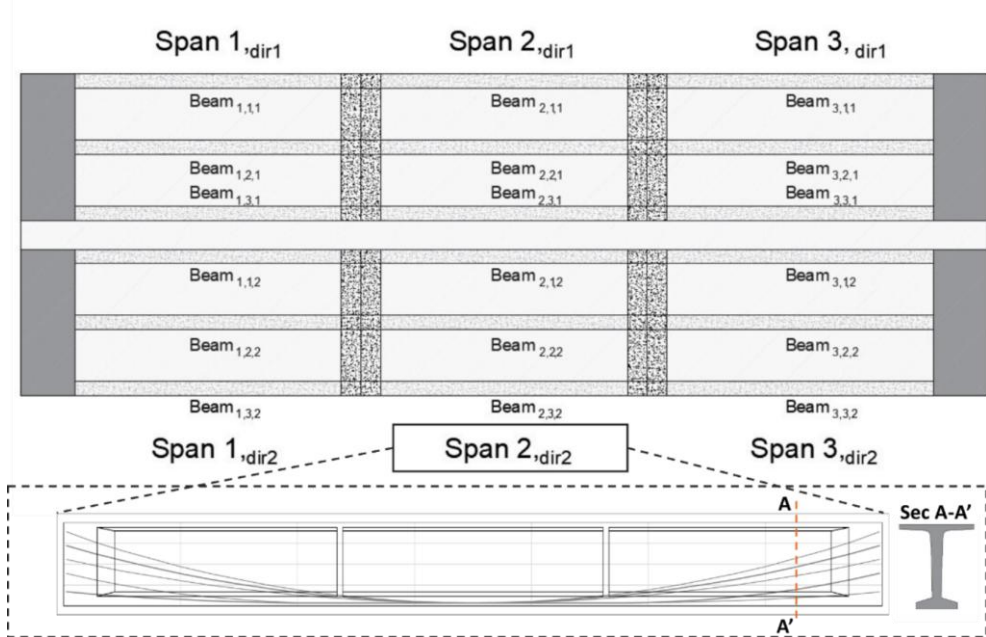


Figure 5-4. Scheme of a typical analysed bridge and layout of cables in longitudinal section.

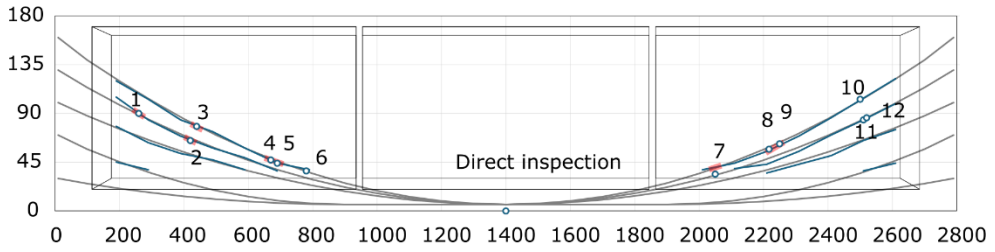


Figure 5-5: Example of the test localisation on a girder. Cable paths shown are the ones detected via GPR (blue) and according to design documents (grey). Red zones indicate the locations of detected voids.

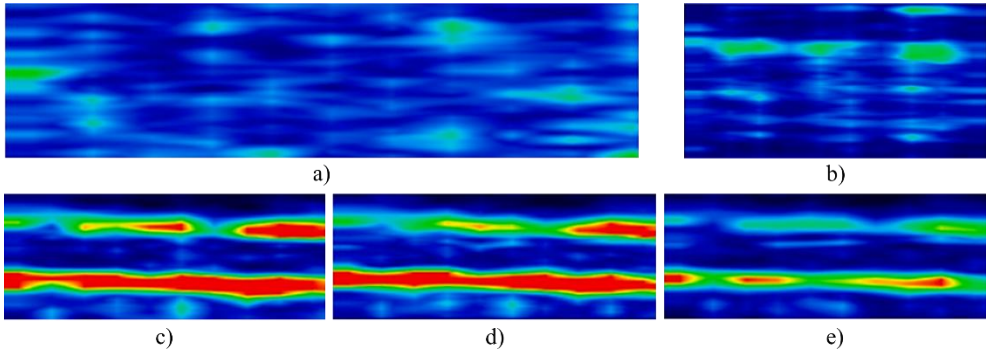


Figure 5-6: UT scans carried out in situ. a) Beam rough surface (honeycombs, b) bottom face of a girder, c) scans performed on the same surface varying the output digital gain.

In the identified critical zones, endoscopic or semi-destructive inspections were then performed to confirm the presence of voids and to detect corrosion (see Figure 5-7). The final dataset includes detailed information on the inspected tendons, the total inspected length, and the precise positions of voids and corrosion spots along each cable. To ensure traceability, each tendon, girder, and bridge is assigned a unique identification code (see Figure 5-4). The resulting dataset serves as the basis for developing probabilistic models of defect occurrence, spatial distribution, and correlations, as presented in the following Sections.

A limitation of the data-gathering procedure is the inability to directly measure the extent of voids and the depth of corrosion due to the lack of access to internal cables. As a result, only the presence of voids or corrosion could be registered, while their extent was qualitatively (visually) assessed during inspections, as also presented in Section 5.3.



Figure 5-7: Data from the inspection campaign. Left: an endoscopic inspection showing a fully grouted and uncorroded cable. Right: a destructive inspection revealing wire corrosion (circled in red) despite the duct being fully grouted.

Therefore, the models presented in Sections 5.4.2 and 5.4.3 account only for the position of corrosion, while severity can only be estimated by incorporating corrosion development models available in the literature (see Chapter 7).

5.3 Statistical analysis

The statistical analysis of the inspection campaign outcomes is conducted on 192 investigated beams, focusing on the distributions of voids and corrosion spots in prestressing wires and strands. The primary objectives are twofold: (i) to examine the correlation between duct-filling defects and corrosion, and (ii) to identify spatial patterns in defect occurrence across different zones of the beam, such as anchorages, curvature changes, and mid-span regions.

To enable statistical evaluation, the inspection results were first qualitatively classified by technicians. Wires were categorised as sound or corroded (slightly or superficially), while duct filling was classified as complete, partial, or nearly void. For consistency in the statistical treatment, fuzzy values are assigned to each class, as reported in Table 5-1.

To explore the relationship between duct-filling defects and corrosion, the conditional probability of corrosion given the void level is analysed (Figure 5-8). The fuzzy values of void and wire corrosion are reported along the horizontal axes, while the vertical axis shows the probability that a cable exhibits a given level of corrosion, conditioned on the void level.

Table 5-1. Definition of fuzzy values.

Duct filling	Void level	Cable conservation	Corrosion level
Complete	1	Sound	1
>75%	2	Slightly corroded	2
50% - 75%	3	Superficially corroded	3
25% - 50%	4		
<25%	5		
(a)	(b)		

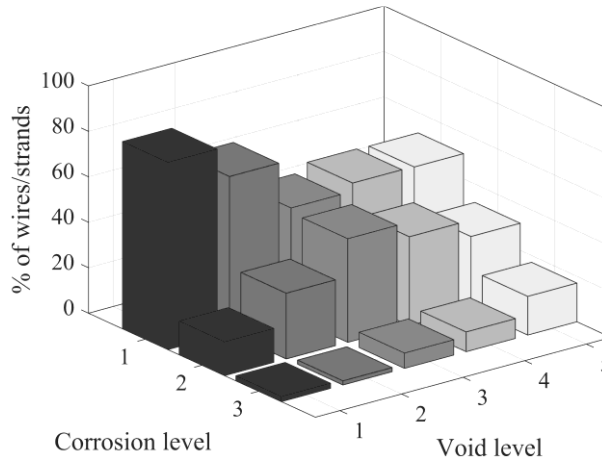


Figure 5-8. Probability of corrosion given the void level.

The results highlight several trends. When duct filling is complete (void level ‘1’), about 70% of wires remain sound, 25% are slightly corroded, and 5% are superficially corroded. This confirms the protective role of complete grout filling, although the presence of some corrosion even in fully filled ducts suggests that additional factors (e.g., grout quality) may also contribute. At void level ‘2’ (>75% filled), the proportion of sound samples decreases, and the probability of corrosion increases. For intermediate fillings (50–75%, void level ‘3’), only 40% of wires are sound, while 35% are slightly corroded, and nearly 10% are superficially corroded. At void levels ‘4’ and ‘5’, superficial corrosion becomes increasingly significant, reaching 25% of cases at the lowest filling.

Overall, Figure 5-8 highlights the strong correlation between void presence and corrosion probability, underlining the importance of incorporating duct-filling conditions into corrosion progression models. From a reliability perspective, void defects act as accelerators of degradation, directly reducing the expected service life of PT tendons.

5.3.1 Spatial distributions of voids and corrosion spots among different zones of the girder

To evaluate whether defects are more likely in specific beam zones, the girders are divided into five regions based on cable inclination and distance from injection points (Figure 5-9). The results are reported with reference to these zones.

The distribution of duct defects (voids) and wire/strand corrosion spots is analysed to assess whether defects concentrate in specific girder zones, such as anchorages, curvature changes, or other areas with difficult grout injection, or whether they are uniformly distributed along the beam and across the cables.

Figure 5-10 depicts the distribution of voids by zone. The results suggest no substantial differences in the extent of voids across zones. However, this apparent uniformity may be influenced by the detection limits of the inspection techniques.

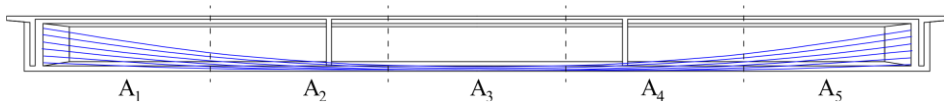


Figure 5-9: Definition of girder zones.

Corrosion levels by zone are reported in Figure 5-11. A slightly worse condition is found at mid-span, where 35% of tendons show level ‘2’ (slightly corroded). This can be interpreted in light of construction practice. Although the standard practice in post-tensioned grouting is to inject grout from the lowest point upward, using vents at high points to expel air and ensure complete filling, historical construction practices may not have consistently achieved these conditions. In older Italian post-tensioned bridges, limited quality control, inadequate venting, injections performed from high points, or interruptions during grouting could have led to incomplete grout filling and local void formation also at the midspan.

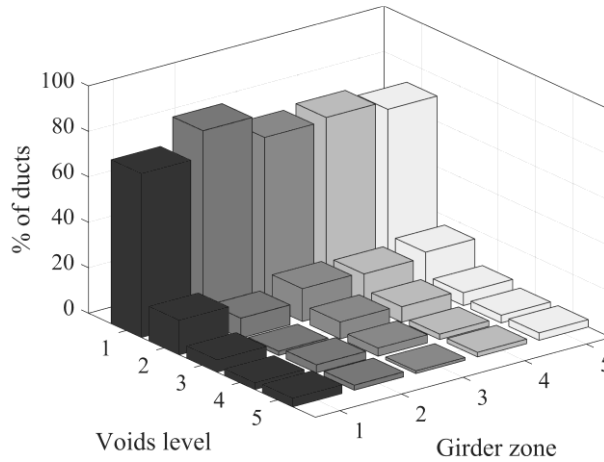


Figure 5-10. Voids level vs zones.

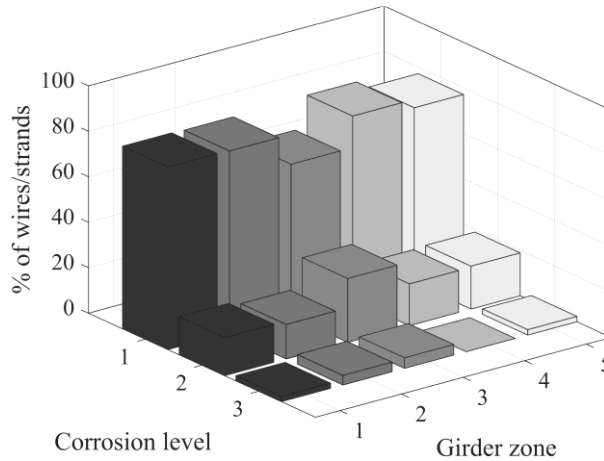


Figure 5-11. Corrosion level vs beam zones.

The influence of girder position within the deck (external vs. internal) is also examined (Figure 5-12). The results show no significant difference, suggesting that environmental exposure is not the primary driver of degradation in this dataset. Instead, defects are mainly linked to construction flaws and tendon layout.

This analysis indicates that the probability of corrosion increases systematically with the severity of void defects, and that even partial voids may compromise tendon durability. Corrosion may also occur, albeit less frequently, in fully grouted ducts. Defects are identified along the entire length of the girders but are more prevalent in zones where grout injection is more challenging (e.g., at curvature changes and

mid-span). By contrast, within the analysed dataset of bridge decks, the beam position within the deck is not found to influence defect occurrence. Therefore, for this specific set of bridges, workmanship and tendon layout are identified as the dominant contributing factors.

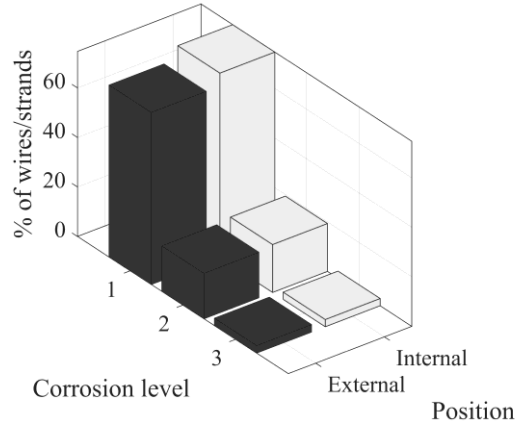


Figure 5-12. Conservation status of wires/strands vs beam position into the deck.

5.4 Probabilistic modelling

The statistical analysis presented in Section 5.3 provides the foundation for tailored reliability assessments of PT bridges [108]. The main objective here is to develop a probabilistic model that characterises: (i) the number of defects present in a girder, (ii) their spatial distribution along the girder length, and (iii) the correlation between voids and corrosion of wires or strands. The model is developed in three steps: first, the probability that a girder contains at least one defect is estimated and incorporated into the time-dependent total probability of failure (Section 5.4.1); then, for defective girders only, conditional distributions for the occurrence rates of corrosion spots and voids, and their joint distribution, are derived (see Section 5.4.2), and the spatial and joint spatial distributions of voids and corrosion are characterised (see Section 5.4.3).

5.4.1 Time-dependent total probability of failure

The inspection of multiple girders can be represented as a sequence of Bernoulli trials, where each girder is considered either defect-free or defective (i.e., containing at least one void or corrosion spot). This assumption is justified because the inspection results naturally lead to binary outcomes at the girder level: a girder is

either unaffected or exhibits at least one defect. Accordingly, the prior probability distribution for a defective girder ($f_D(e)$) can be expressed as:

$$f_D(e) = e \cdot \theta \quad (20)$$

where $e = 0$ indicates no defects, $e = 1$ indicates the presence of at least one defect, and θ represents the unknown probability of defect presence.

To estimate the parameter θ , the Bayesian approach proposed by Faber and Sørensen [100] is applied, using a non-informative prior combined with the binomial likelihood function:

$$L_D(n_D|\theta) = \binom{n}{n_D} \theta^{n_D} (1 - \theta)^{n - n_D} \quad (21)$$

where $L_D(n_D|\theta)$ represents the likelihood function, n_D is the number of defective girders, n the total number of girders inspected. The posterior distribution of θ ($f''_{\theta}(\theta|n_D)$) is then:

$$f''_{\theta}(\theta|n_D) = \frac{\binom{n}{n_D} \theta^{n_D} (1 - \theta)^{n - n_D}}{c} \quad (22)$$

where c is a normalisation constant ensuring the posterior integrates to one. The predictive distribution for a defective girder can then be defined using the posterior probability density f''_{θ} :

$$f_D(e|n_D) = \int_0^1 f_D(e) \cdot f''_{\theta}(\theta|n_D) d\theta = \int_0^1 e \cdot \theta \cdot f''_{\theta}(\theta|n_D) d\theta \quad (23)$$

This distribution can be interpreted as a Bernoulli model with parameter $\int_0^1 \theta \cdot f''_{\theta}(\theta|n_D) d\theta$ (denoted as $P_{\mathbb{D}}$ hereafter), which represents the probability of defect presence after the integration of statistical uncertainties. The integral in Eq. (23) can be solved in closed form and, for the 192 inspected girders, of which 140 are found to contain defects, the posterior probability of defect presence is evaluated as follows:

$$P_{\mathbb{D}} = \frac{\Gamma(n + 2)}{\Gamma(a)\Gamma(n - n_D + 1)} \frac{\Gamma(a + 1)\Gamma(b + 1)}{\Gamma(a + b + 2)} = 0.7268 \quad (24)$$

with $a = n_D + 1$ and $b = (n + 1) - (n_D + 1)$. Defining the event $\mathbb{D}$ as: $\mathbb{D}: g_{\mathbb{D}} = 1 - e \leq 0$ the value of $P_{\mathbb{D}}$ can be incorporated into reliability analysis using the total probability theorem to evaluate the overall probability of failure of a girder at time t :

$$P(F(t)) = P_{\mathbb{D}} \cdot P(F(t)|\mathbb{D}) + (1 - P_{\mathbb{D}}) \cdot P(F(t)|\bar{\mathbb{D}}) \quad (25)$$

where $P(F(t))$ is the total probability of failure at time t , $P(F(t)|\mathbb{D})$ is the conditional probability of failure at time t given that the girder is defective (event $\mathbb{D}$), and $P(F(t)|\bar{\mathbb{D}})$ is the conditional probability of failure at time t given that the

girder is not defective (event $\bar{\mathbb{D}}$). This formulation is consistent with the general framework introduced in Chapter 4 and ensures that system reliability reflects both the probability of defect occurrence and its influence on structural degradation.

5.4.2 Rate of occurrence of a defect per meter of girder inspected

To determine the defect occurrence rate (defined as the number of defects per metre of inspected cable), only cables containing at least one void or corrosion spot are considered. The inspected length of each cable is estimated from endoscopy-examined sections (where tomography was also performed) and from destructively inspected sections, typically at mid-span, of which the inspected length is normalised to one metre. The total inspected length of each girder is obtained by summing the individual cable lengths, and the defect occurrence rate is calculated as the ratio between the total number of detected defects (voids or corrosion spots) and the inspected length. Cable inclinations are disregarded, as their influence on the results is negligible.

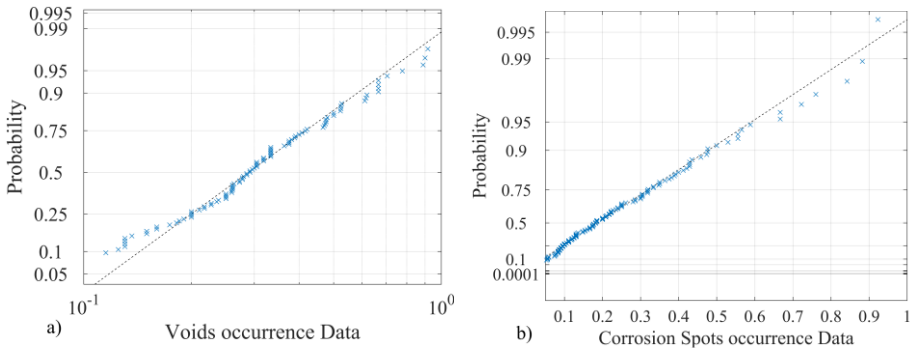


Figure 5-13: Probability plots of the occurrence rate per metre of inspected girder for a) voids and b) corrosion spots, given a defective girder.

A goodness-of-fit test is performed to identify suitable probability distributions for defect occurrence rates. The Cramer-Von Mises test [109] is selected because of its robustness in evaluating the overall difference between empirical and theoretical cumulative distribution functions (CDFs). The test indicated that a Lognormal distribution best represents the void occurrence rate ($\rho_V|\mathbb{D}$), while a Gamma distribution best represents the corrosion spot occurrence rate ($\rho_C|\mathbb{D}$). Figure 5-13a and b, where the X-axis represents the occurrence of voids, and the Y-axis shows the probability, provide probability plots for the two variables. In both cases, the empirical probabilities (cross symbols) are closely aligned with the theoretical probabilities (dashed lines), confirming the good fit of the selected models.

The analysis shows that the void occurrence rate can be modelled using a lognormal distribution. However, the UT scans employed to detect voids are subject to detection uncertainties that depend on void size. These uncertainties are accounted for using the UT-PoD curve developed in [46]. First, the empirical probability mass function (PMF) of void size is derived from in situ UT scan data using a frequentist approach. Void lengths are grouped into classes of width 10 cm (corresponding to the UT scan spacing), and the number of detected voids in each class c_i , denoted as $C_{v,data}(c_i)$, is counted. Because the PoD decreases for small voids, the raw counts underestimate the occurrence of small voids. To compensate, each class count is corrected using the average PoD value over that class, resulting in the PoD-adjusted class count, $C_{v,data}^{adj}(c_i)$:

$$C_{v,data}^{adj}(c_i) = \frac{C_{v,data}(c_i)}{\overline{PoD}_{c_i}} = \frac{C_{v,data}(c_i)}{\frac{1}{10} \int_{c_i} PoD(v) dv} \quad (26)$$

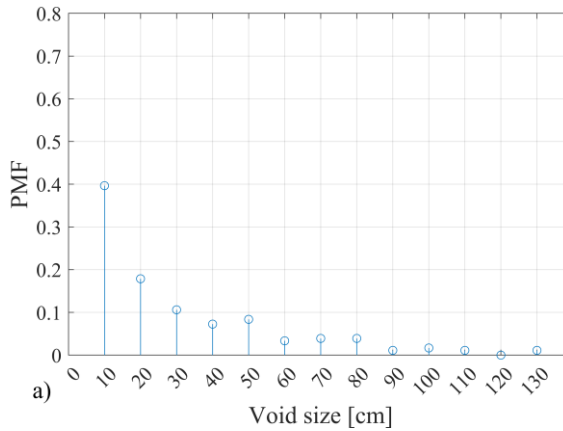
where $\overline{PoD}_{c_i}$ is the average probability of detection of voids within class c_i , $PoD(v)$ is the detection probability function depending on void size v , and 10 cm is the class width.

The adjusted PMF of the void size, $p_{v,data}^{adj}(c_i)$, is then estimated as:

$$p_{v,data}^{adj}(c_i) = \frac{C_{v,data}^{adj}(c_i)}{\sum_{c_i} C_{v,data}^{adj}(c_i)} \quad (27)$$

where $\sum_{c_i} C_{v,data}^{adj}(c_i)$ represents the PoD-adjusted total number of detected voids.

This procedure compensates for the higher miss rate of smaller voids. Figure 5-14 illustrates the recorded a) and PoD-adjusted b) size PMFs.



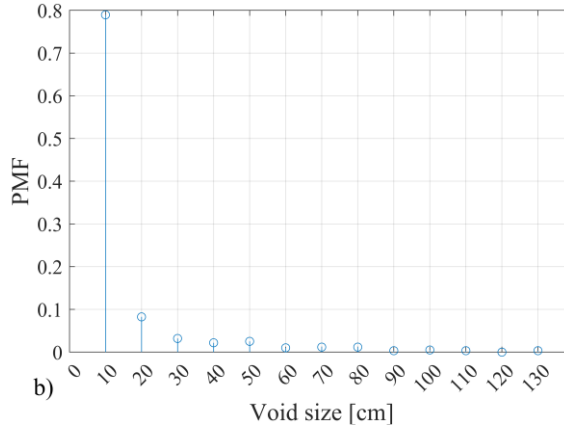


Figure 5-14: Probability mass function of recorded a) and PoD-adjusted b) void sizes.

By modelling the void size uniformly distributed within each class, the unconditional probability of indication during a tendon inspection can be expressed as the expectation of the PoD function over the adjusted size distribution:

$$P(I) = \int PoD(v) \cdot p_{v_{data}}^{adj}(v) dv \quad (28)$$

Using the acquired data and the PoD curve provided in [46], the unconditional probability of void indication is estimated as $P(I) = 0.299$.

Finally, the adjusted number of voids for each girder O_j^{adj} is computed as $O_j^{adj} = O_j / P(I)$, where O_j is the number of observed voids for girder j . The void occurrence rate variable ($\rho_V | \mathbb{D}$), is then modelled by fitting a lognormal distribution to the occurrence rates calculated using the adjusted number of voids.

5.4.2.1 Joint rate of occurrence of voids and corrosion spots

The correlation between void and corrosion occurrence rates, $\rho_V | \mathbb{D}$ and $\rho_C | \mathbb{D}$, is analysed. The Pearson correlation coefficient is 0.3940 (p-value = 0.0000; i.e., the probability of incorrectly assuming a correlation when none exists), while the Kendall-Tau association coefficient is 0.2471 (p-value = 0.0002), both indicating a statistically significant correlation. This result is consistent with engineering expectations and the statistical analysis presented in Section 5.3.

To capture the dependence between $\rho_V | \mathbb{D}$ and $\rho_C | \mathbb{D}$, copula functions are employed. The copula approach allows for combining the separate marginal distributions into a joint distribution ($\rho_{CV} | \mathbb{D}$) accounting for their dependence, as defined in [26]. After bootstrap-based goodness-of-fit evaluations for several copula families, as described in [28] and [27], the centred- χ^2 -copula is selected based on the minimum squared difference between the empirical copula and the selected copula family.

This choice is justified because the centred- χ^2 -copula effectively captures asymmetric tail dependence, as discussed in [110].

The quality of the joint model is evaluated using the Bhattacharyya coefficient [111], which measures similarity between two probability density functions. It is computed by taking the square root of the product of the density values of the two distributions at each point and summing the results. The coefficient ranges from 0 to 1, where higher values indicate greater similarity; 1 corresponds to identical distributions. Fifty simulated datasets, generated from the joint distribution $\rho_{CV}|\mathbb{D}$, are compared with the observed bivariate sample. Bhattacharyya coefficient ranged from 0.97 to 0.99, indicating good agreement between the simulated and observed data. The parameters for the joint defect rate-of-occurrence model are shown in Table 5-2.

Table 5-2: Calibrated probabilistic model for corrosion spots and voids rate of occurrence.

Variable	Marginal Distribution	Mean	Standard deviation	Copula family	Copula parameter
$\rho_{CV} \mathbb{D}$	$\rho_C \mathbb{D} \sim \text{Gamma}$	0.2615	0.1905	Centred- χ^2	$\theta_{co} = 0.3940$
	$\rho_V \mathbb{D} \sim \text{Log-Normal}$	1.0787	0.7568		

5.4.3 Spatial distribution of voids and corrosion spots

The variation in the probability of detecting a defect along the girder length is analysed to identify zones that are more prone to grout voids or corrosion. In particular, Figure 5-15 illustrates the probability (Y-axis) of detecting either a void or a corrosion spot at a given location, normalised by the number of inspections performed at that location, as a function of the relative position along the girder length (X-axis).

The results show a trimodal pattern for both voids and corrosion spots, indicating that defects are concentrated in three main regions: near the girder ends, around mid-span, and at curvature changes. This outcome is consistent with construction practices, as grout injection is often less effective at anchorages, curvature changes, and mid-span regions.

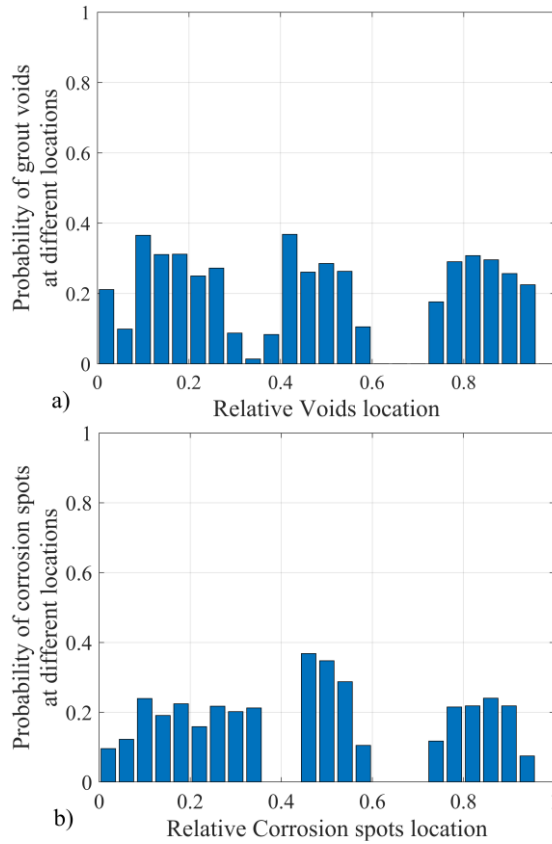


Figure 5-15: Probability of detecting a grout void a) or a corrosion spot b) along the girder length, given a defective girder.

According to Figure 5-15, parametric trimodal normal distributions are fitted to the data, resulting in two probabilistic models, $\zeta_V|\mathbb{D}$ and $\zeta_C|\mathbb{D}$, which represent the spatial distribution of voids and corrosion spots, respectively. Given the symmetric tendon layout of the girders and the arbitrary selection of starting and ending points during data recording, the fitted distributions are constrained to be symmetric around the midpoint (position 0.5). This constraint reflects the physical symmetry of the girder layout, ensuring that the statistical model is consistent with the structural configuration. Goodness of fit is evaluated using probability plots, as shown in Figure 5-16. For voids (Figure 5-16a), empirical probabilities (crosses) align closely with the theoretical values derived from the fitted distribution (dashed line), confirming the accuracy of the model. A similar comparison for corrosion spots (Figure 5-16b) shows that $\zeta_C|\mathbb{D}$ also provides a good representation of the observed data.

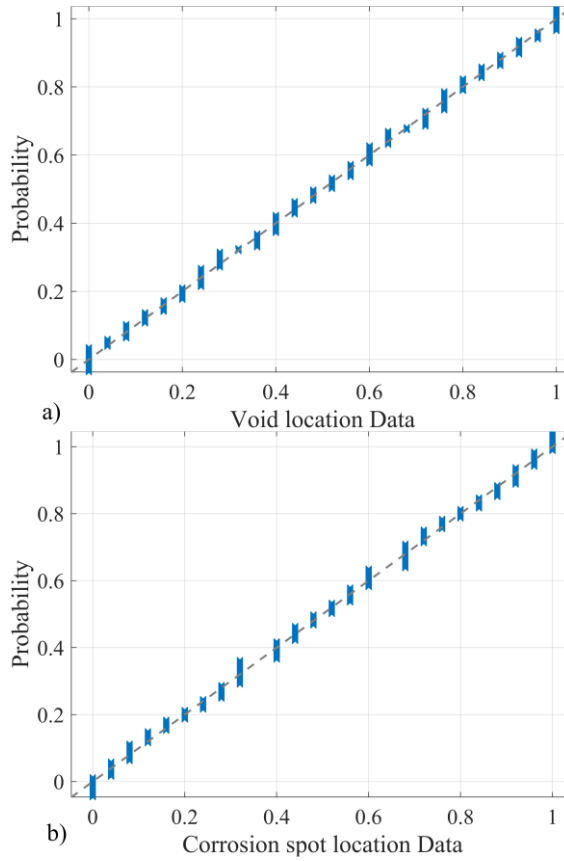


Figure 5-16: Probability plots of void locations a) and corrosion spot locations b) for defective girders.

5.4.3.1 Joint spatial distribution of voids and corrosion spots

The joint spatial distribution of voids and corrosion spots is modelled using the same procedure adopted in Section 5.4.2.1. The Pearson correlation coefficient (0.4521, p-value = 0.0000) and the Kendall-Tau coefficient (0.4101, p-value = 0.0000) both confirm a statistically significant correlation between the spatial locations of voids and corrosion. This implies that voids and corrosion tend to cluster in the same girder regions, reinforcing the physical interpretation that poorly injected zones are more susceptible to both defects. Among several candidate copula families (evaluated using bootstrap-based goodness-of-fit procedures, see Section 5.4.2.1), the Gaussian copula is found to provide the best representation of the dependence structure. This model is used to describe the joint spatial distribution of voids and corrosion spots ($\zeta_{CV}|\mathbb{D}$) along the girder length. As in Section 5.4.2.1, the joint fit quality is assessed using the Bhattacharyya coefficient [111]. Across fifty

simulations, values between 0.85 and 0.97 are obtained, confirming the adequacy of the joint model. The joint model parameters are documented in Table 5-3.

Table 5-3: Calibrated probabilistic model for the joint spatial occurrence of corrosion spots and voids.

Variable	Marginal Distribution	Mean	Standard deviation	Component proportions	Copula family	Copula parameter
$\zeta_{CV} \mathbb{D}$	$\zeta_C \mathbb{D} \sim \text{Trimodal Normal } [0, L_{girder}]$	$\begin{bmatrix} 0.18 ; 0.50 \\ ; 0.82 \end{bmatrix} \cdot L_{girder}$	$\begin{bmatrix} 0.1 ; 0.06 \\ ; 0.1 \end{bmatrix} \cdot L_{girder}$	$\begin{bmatrix} 1/3 ; \\ 1/3 ; 1/3 \end{bmatrix}$	Gaussian	$\theta_{co} = 0.45$
	$\zeta_V \mathbb{D} \sim \text{Trimodal Normal } [0, L_{girder}]$	$\begin{bmatrix} 0.14 ; 0.50 \\ ; 0.86 \end{bmatrix} \cdot L_{girder}$	$\begin{bmatrix} 0.08 ; 0.09 \\ ; 0.08 \end{bmatrix} \cdot L_{girder}$	$\begin{bmatrix} 0.35 ; 0.31 \\ ; 0.35 \end{bmatrix}$		

5.5 Summary of the Chapter

The probabilistic models for injection defects in PT girders are developed using inspection data from 192 girders across 53 bridges. The models capture both the occurrence rates and the spatial distributions of voids and corrosion spots, along with their correlations. Notably, the development of the voids occurrence rate model accounts for voids detection uncertainties by using the UT-PoD curve to both adjust the void size distribution and estimate the unconditional probability of void indication.

The analysis confirmed that voids and corrosion are positively correlated and tend to cluster in specific girder regions, particularly at anchorages, curvature changes, and mid-span. These patterns reflect construction practices and tendon layout, with voids serving as preferential pathways for aggressive agents and as accelerators of tendon degradation by facilitating corrosion.

From a system reliability perspective, these results are highly relevant, as the presence of voids accelerates corrosion, which in turn reduces the reliability of the girder system as a whole. The models derived here provide the necessary input parameters for the system reliability framework introduced in Chapter 4.

In Chapter 6, these models are embedded into the framework through a simulation procedure to estimate the spatial and time-dependent reliability of PT girders and decks.

6 Procedure for performing Reliability Assessment of PT Bridges with Spatially Distributed Defects

To address RQ3.c, the probabilistic defect model developed in Chapter 5 is applied here to account for the spatial variability of the corrosion process in PT bridges when evaluating their reliability. In this formulation, both the number of degrading sections and their positions are treated as random variables. As a result, multiple degradation scenarios can be simulated by varying the number ($n_{i,k}$) and locations of sections subject to deterioration. Each degrading section may either (i) develop corrosion in isolation (i.e., a corrosion spot) or (ii) experience corrosion in the presence of an injection void. In the latter case, corrosion is modelled to initiate earlier to incorporate the evidence that corrosion is accelerated in the presence of voids (see also Sections 5.3 and 7.3).

To perform the calculations, an advanced simulation procedure involving n_{MCS} iterations is proposed, where n_{MCS} denotes the total number of simulated scenarios. Each iteration corresponds to a distinct configuration of the girder's series system, defined by a unique set of defect numbers and locations. This approach represents a departure from traditional deterministic modelling of defects, allowing the system reliability analysis to reflect both the stochastic nature of defect occurrence and their spatial clustering.

In each k-th iteration of the simulation, the number of corrosion spots and voids is determined by sampling from the joint occurrence distribution $\rho_{CV}|\mathbb{D}$ (Table 5-2), yielding values for corrosion spots per metre (r_C) and voids per metre (r_V). The absolute numbers of corrosion spots and voids are then computed as $n_C = L_{girder} \cdot r_C$ and $n_V = L_{girder} \cdot r_V$, where L_{girder} is the length of the girder. The maximum of these two values, $n = \max(n_C; n_V)$, defines the number of potential defect positions to be generated.

The spatial positions of defects are obtained by sampling n realisations from the joint spatial distribution $\zeta_{CV}|\mathbb{D}$ (Table 5-3), providing vectors of relative positions

for corrosion spots (s_C) and voids (s_V). Surplus positions are trimmed so that $s_C = n_C$ and $s_V = n_V$. These relative positions are converted into absolute coordinates by multiplying by L_{girder} . The overlap between voids and corrosion spots is then identified by comparing their distances from the girder end.

From this procedure, three position vectors are defined: S_C positions containing corrosion spots only; S_V positions containing voids only (excluded from reliability analysis, as no corrosion develops there); and S_{CV} positions where voids and corrosion coincide. For each iteration k , the sets of active degrading positions ($x_{C,k}$) are stored in $S_{C,k}$, while positions experiencing corrosion in the presence of a void ($x_{CV,k}$) are stored in $S_{CV,k}$. The total number of analysed sections, $n_{i,k}$, is then obtained as the sum of the lengths of these vectors. The corresponding absolute positions, $x_{C,k}$ and $x_{CV,k}$, are used in the reliability analysis (Figure 6-1).

By repeating this process across n_{MCS} iterations, a large ensemble of possible system configurations is generated, each reflecting a different spatial distribution of defects. The structural response is then computed for each scenario, with load effects at individual sections evaluated through influence lines. This procedure ensures that the reliability analysis explicitly incorporates the stochastic and spatially dependent nature of PT girder degradation, providing a potentially more realistic and relevant estimate of service life and safety.

The main benefit of the proposed simulation method is that it treats both the corroding sections and their spatial positions as random variables. In doing so, it overcomes the traditional assumption of a uniform corrosion length and provides a representation that is better aligned with the actual, spatially heterogeneous condition of existing structures.

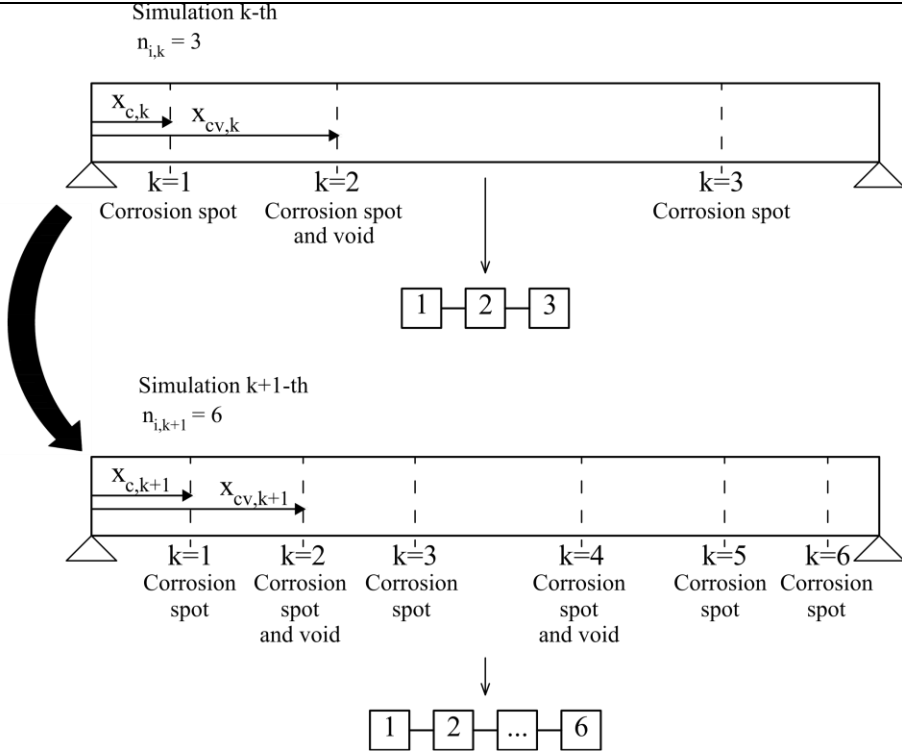


Figure 6-1: Illustration of two simulation iterations showing the random number and positions of corrosion spots and voids, leading to distinct system configurations.

This procedure is consistent with the LSFs defined in Chapter 4 and can therefore be directly embedded into the reliability framework. For any simulation iteration and given the event $\mathbb{D}$, Eq. (7) can be adapted as follows:

$$g_{f,i,k}(\mathbf{X}, \mathbf{S}, t) = g_{F(t)|\mathbb{D},k}(\mathbf{X}, \mathbf{S}, t, k, \varsigma_{i,k}) \\ = \theta_{f,C} C_f(\mathbf{X}, t, k, \varsigma_{i,k}) - \theta_{f,D} D_{f,\max(t)}(\mathbf{S}, k, \varsigma_{i,k}) \quad (29)$$

where $g_{F(t)|\mathbb{D},k}(\mathbf{X}, \mathbf{S}, t, k, \varsigma_{i,k})$ is the limit state function evaluated at section $\varsigma_{i,k}$ at time t , conditioned on event $\mathbb{D}$; $C_{f,i}(\mathbf{X}, t, k, \varsigma_{i,k})$ is the capacity corresponding to the f -th failure mode of the section $\varsigma_{i,k}$ at time t ; and $D_{f,i,\max(t)}(\mathbf{S}, k, \varsigma_{i,k})$ is the demand corresponding to the f -th failure mode at the section $\varsigma_{i,k}$ calculated using its influence line. Following Eq. (29), all other LSFs presented in Chapter 4 can be adapted to account for spatially random defect distributions.

The failure probability at time t for a sound structure, $P(F(t)|\bar{\mathbb{D}})$, is calculated without considering tendon degradation due to hidden defects. The total failure

probability ($P(F(t))$) is then evaluated using Eq. (25), and the corresponding total reliability index (β) is defined as:

$$\beta = -\Phi^{-1}[P(F(t))] = -\Phi^{-1}[P_{\mathbb{D}} \cdot P(F(t)|\mathbb{D}) + (1 - P_{\mathbb{D}}) \cdot P(F(t)|\overline{\mathbb{D}})] \quad (30)$$

In Eq. (30), $-\Phi^{-1}$ represents the inverse cumulative function of the Standard Normal Distribution and all other symbols retain the meanings introduced previously.

6.1 Summary of the Chapter

In this chapter, the probabilistic models of defect occurrence and spatial distribution developed in Chapter 5 are embedded into the system reliability framework introduced in Chapter 4. An advanced simulation procedure is proposed, in which both the number and positions of degrading sections are treated as random variables; thus, reducing conservativeness in the assumption of the defect location and providing a solution to overcomes the segmentation length depending solution of such problems.

The incorporation of defect probabilities, occurrence rates, and spatial correlations provides a potentially more realistic and relevant estimate of service life than uniform degradation models and enables direct use of inspection data. Although computationally demanding, the method remains fully consistent with the limit state formulations and can be adapted to large-scale applications through advanced sampling techniques.

Overall, the procedure helps bridge the gap between empirical evidence of defects and structural reliability assessment, and it sets the stage for the case study applications presented in Chapter 8.

The following Chapter 7 focuses on the modelling of corrosion initiation and propagation processes within the reliability framework.

7 Corrosion Degradation Modelling

This Chapter addresses explicitly RQ2 by providing the models and methods for integrating corrosion-induced degradation under different concrete conditions within the proposed system reliability framework.

As discussed in Chapter 2, both generalised and localised corrosion processes depend on the quality of the concrete surrounding the reinforcement or tendons, as well as on the presence of aggressive agents such as chlorides or on the reduction of alkalinity within the cover.

Therefore, in this Chapter, the interplay between alkalinity loss and chloride ingress is represented through logical system modelling, which enables an explicit representation of how concrete quality influences the initiation of both generalised and localised corrosion. In this work, alkalinity reduction is represented by concrete carbonation, and Figure 7-1 illustrates the logical systems used to model the onset of the two corrosion processes.

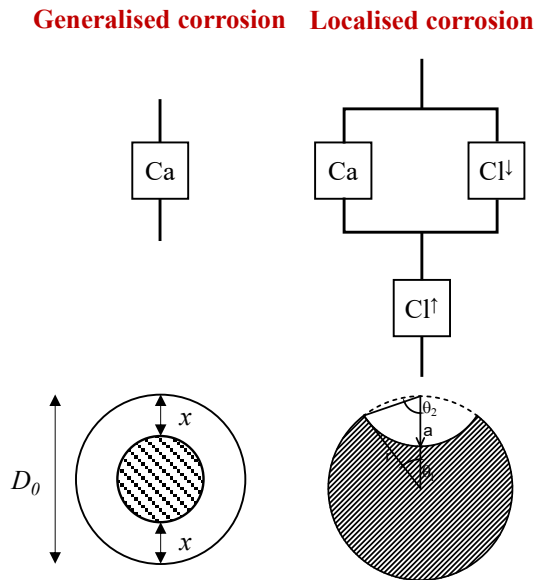


Figure 7-1: Logical system representation of the activation of generalised and localised corrosion.

In the figure, Ca denotes the progression of carbonation, while Cl^l and Cl^h indicate the lower or higher chloride concentrations required to trigger localised corrosion, depending on concrete quality. At the bottom of the figure, the geometry of the steel sections subjected to corrosion is schematically shown: D_0 is the initial diameter, x is the uniform diameter reduction associated with generalised corrosion, and a is the pit depth associated with localised corrosion.

This system representation offers two main advantages. First, it formalises the relationship between carbonation (i.e., alkalinity reduction) and chloride ingress, explicitly representing their interaction in the initiation of corrosion. Second, it distinguishes between uniform and pitting attack, allowing the subsequent propagation models to reflect the nature of corrosion processes.

7.1 Activation phase

As illustrated in Figure 7-1, the initiation of generalised corrosion is primarily attributed to carbonation [75], with the activation time modelled as the period required for the carbonation front to reach the reinforcement depth. Conversely, the initiation of localised (pitting) corrosion is associated with chloride ingress, and the activation time is defined as the time needed for the chloride concentration at reinforcement depth to exceed the critical chloride threshold (C_{cr}). The value of C_{cr} is uncertain and strongly influenced by the quality of the surrounding concrete. To reflect this variability, two probability distributions are defined: a distribution with a higher mean value (Cl^h in Figure 7-1) for sound concrete (C_{cr_s}), i.e. concrete that maintains high alkalinity; a distribution with a lower mean value (Cl^l in Figure 7-1) for carbonated concrete (C_{cr_c}), where alkalinity has been reduced. This model aims to replicate the phenomenological model described in [112], as well as the findings of the thorough review by Angst et al. [76], which showed that alkalinity has a major influence on the pitting potential and, consequently, on the chloride concentration required to initiate corrosion. This interpretation is further supported by [73], where corrosion was not observed in existing concrete structures despite very high chloride ion concentrations. When carbonation is modelled explicitly, the distribution C_{cr_c} is applied in regions where the carbonation front has reached the reinforcement depth. The activation event is formalised using the following LSFs:

$$g_{g,i} = t - T_{g,i}(M_{ca}(\Theta_{ca})) \quad (31)$$

$$g_{p,i} = \max \left\{ \begin{aligned} &\left[t - T_{p,i}(M_{ch,s}(\Theta_{ch})) \mid T_{g,i}(M_{ca}(\Theta_{ca})) \geq T_{p,i}(M_{ch,c}(\Theta_{ch})) \right] \\ &\left[t - T_{p,i}(M_{ch,c}(\Theta_{ch})) \mid T_{g,i}(M_{ca}(\Theta_{ca})) < T_{p,i}(M_{ch,c}(\Theta_{ch})) \right] \end{aligned} \right\} \quad (32)$$

$$g_{c,i} = \max\{g_{g,i}; g_{p,i}\} \quad (33)$$

where $g_{g,i}$ is the LSF for the initiation of generalised corrosion in any section i , $T_{g,i}(M_{ca}(\Theta_{ca}))$ is the time to generalized corrosion, depending on the carbonation model M_{ca} and the relative parameters Θ_{ca} , $g_{p,i}$ is the LSF for the initiation of pitting corrosion in any section i , $T_{p,i}(M_{ch,s}(\Theta_{ch}))$ and $T_{p,i}(M_{ch,c}(\Theta_{ch}))$ are the times to pitting corrosion in sound and carbonated concrete, respectively, depending on the chloride diffusion model $M_{ch,s}$ or $M_{ch,c}$ and parameters Θ_{ch} . Finally, $g_{c,i}$ is the governing LSF for corrosion initiation. When $g_{c,i} > 0$, corrosion is considered to have initiated in section i (event $\mathbb{C}_i$), consistently with Eq. (8) in Section 4.3.

7.1.1 Carbonation model

Carbonation depth is commonly described as a function of the square root of time multiplied by a carbonation coefficient, which reflects the influence of concrete mixture design, chemical composition, and environmental conditions, [66], [113], [114], [115]. The time to generalised corrosion, T_g , can then be evaluated as follows:

$$T_g(M_{ca}(\Theta_{ca})) = \frac{c_c^2}{k^2} \quad (34)$$

where c_c is concrete cover [mm] and k is the carbonation parameter. A prior distribution p'_k can be defined for the carbonation parameter, based on the selected carbonation model and available information. In order to define the initial (a priori) probabilistic characterisation of the carbonation parameter k , the model proposed by Silva et al. [115] is adopted. This model provides the statistical description required as input to the updating procedure. Throughout the thesis, the model by Silva et al. [115] is therefore employed to define the a priori distribution of the carbonation parameter. The square-root law is then adopted to calculate T_g because it is widely supported by experimental evidence, ensuring non-negative carbonation depths and reflecting the diffusion-driven nature of the process.

7.1.1.1 Updating of the Carbonation Model and Time to Generalised Corrosion

As discussed in Section 4.4, direct modelling of the process error for carbonation testing is challenging, particularly in specifying the standard deviation σ_ϵ associated with measurement uncertainties. For this reason, the updating procedure is applied directly to the model parameter. In particular, test data provide estimates of the carbonation coefficient from measured depths:

$$\Theta_{ca_r} = k_r = \frac{d_r}{\sqrt{t_r}} \quad (35)$$

where d_r is the measured carbonation depth from the r -th test, t_r is the corresponding test year, and k_r the derived carbonation parameter.

Several candidate probability distributions, such as Exponential, Lognormal, Generalised Extreme Value, Gumbel, Normal, and Gamma, can be fitted to the dataset of estimated k_r values. The best-fitting distribution may then be selected using a goodness-of-fit test, such as the Cramér-von Mises criterion [116], [117]. The selected fitted distribution (p_k) is then adopted as the likelihood for updating the carbonation coefficient:

$$L(o_1, \dots, o_n | \Theta_{ca}) = L(d_1, \dots, d_n | k) = p_k(\lambda_k, \xi_k) \quad (36)$$

where λ_k, ξ_k are the parameters of the fitted distribution. Referring to Eq. (14), the posterior distribution can be computed numerically as:

$$p''(\Theta_{ca} | d_1, \dots, d_n) = p_k''(k) = \frac{1}{c} L(d_1, \dots, d_n | k) \cdot p_k'(k) \quad (37)$$

where c is the normalisation constant and $p_k'(k)$ is the prior distribution of the carbonation parameter. The posterior distribution of the carbonation parameter can be used to evaluate the updated time to generalised corrosion (Eq. (34)).

7.1.2 Chloride diffusion model

Chloride diffusion models are typically founded on Fick's second law. Several models can be found in the literature, each tailored to accommodate varying boundary conditions and environmental factors, as documented in [68]. Among the parameters influencing corrosion activation, the apparent surface chloride concentration ($C_{s,app}$) and the chloride diffusion coefficient ($D_{RCM,0}$) have a significant impact on the predicted time to corrosion and usually require dedicated characterisation. Because of the significant uncertainties associated with $C_{s,app}$, its mean value ($\mu_{C_{s,app}}$) is commonly treated as a random variable.

The time to pitting corrosion ($T_p(M_{ch}(\Theta_{ch}))$) is modelled as a parameterised stochastic process derived from Fick's second law of diffusion, which describes the buildup of chloride concentration at the steel cover depth (c_c). The expression is given as [67]:

$$T_p(M_{ch}(\Theta_{ch})) = K_c \left[\frac{1}{4D_{app}} \left(\frac{c_c}{\text{erf}^{-1}\left(1 - \frac{C_{cr}}{C_{s,app}}\right)} \right)^2 \right] \quad (38)$$

where K_c represents model uncertainty in the chloride diffusion process, c_c is the concrete cover [mm], C_{cr} is the critical chloride concentration [wt.-%/b], $C_{s,app}$ is the apparent surface chloride concentration [wt.-%/b], k_e is the environmental function [67], D_{app} is the apparent chloride diffusion coefficient [mm²/year], which depends on $D_{RCM,0}$ [67]. Both $C_{s,app}$ and D_{app} are referred to as “apparent”

parameters because they do not represent intrinsic chemical properties of diffusion. Instead, they are obtained by calibrating Fick's law against measured chloride profiles in concrete. This calibration process serves as the basis for defining the probability distributions of $C_{s,app}$ and D_{app} in [65], [67], making these references appropriate for describing the process. Finally, because Eq. (38) relies on empirically derived parameters, the variable K_c plays a key role in capturing residual model uncertainty. Following [52], it is modelled as a normally distributed random variable with a mean of 1.0 and a coefficient of variation of 0.1.

7.1.2.1 Updating Chloride Ingress Models for Predicting Pitting Corrosion Initiation

When chloride concentration data are available from inspections or laboratory tests, it becomes possible to update the chloride diffusion model and, subsequently, the predicted time to pitting corrosion. In this case, the inverse function of the diffusion model is not analytically defined. Therefore, the likelihood function $L(o_1, \dots, o_n | \Theta_{ch})$ is obtained directly from the calibration process (see Section 4.4). A key requirement of the updating procedure is the preservation of the temporal characteristics of the diffusion process. This is essential, since prior models are based on established physical principles that describe how chloride ingress evolves over time. Updating should therefore refine the parameter estimates using empirical evidence, without altering the fundamental time-dependent nature of the process.

A non-linear regression analysis can be conducted to fit the model based on Fick's second law of diffusion (Eq. (39)) to the experimental data. The chloride concentration at depth d at time t can be expressed as:

$$\begin{aligned} M_{ch}(\Theta_{ch}, d, t) &= Cl_{mo}(d, t) \\ &= C_0 + (C_{s,app} - C_0) \left[1 - \operatorname{erf} \left(\frac{d - \Delta d_c}{2\sqrt{D_{app} \cdot t}} \right) \right] \end{aligned} \quad (39)$$

where C_0 [wt.-%/c] is the initial chloride concentration and Δd_c is the depth of the convection zone [mm].

The parameters adjusted for the regression process are the apparent chloride concentration, $C_{s,app}$, and the apparent chloride diffusion coefficient, $D_{app,C}(\Theta_{ch} = [C_{s,app}; D_{app,C}])$. These are chosen for their physical and model relevance, as well as numerical stability. For a fixed time t , the model is fitted to the observed data using a non-linear least squares approach:

$$Cl_{mo}(d, \bar{t}) \approx Cl_{mo,reg}(d, \bar{t}) = f_{Cl_{mo}}([\bar{d}, \bar{t}], \Theta_{ch}) + \varepsilon \quad (40)$$

In Eq. (40), $Cl_{mo}(d, \bar{t})$ represents the observed data, i.e., the chloride concentration, while $f_{Cl_{mo}}([\bar{d}, \bar{t}], \Theta_{ch})$ is the model function introduced in Eq. (39). $[\bar{d}, \bar{t}]$ is the predictor matrix (depth and inspection year), and ε is the error term. The standard

model for ε is applied, i.e., it has zero mean, standard deviation σ_ε , and it follows a Normal distribution. The error variance is estimated as:

$$\sigma_\varepsilon^2 = \frac{\left(\mathbf{cl}_{mo}(\bar{d}, \bar{t}) - \widehat{\mathbf{cl}_{mo}}(\bar{d}, \bar{t}) \right)^T \left(\mathbf{cl}_{mo}(\bar{d}, \bar{t}) - \widehat{\mathbf{cl}_{mo}}(\bar{d}, \bar{t}) \right)}{n - p} \quad (41)$$

where $\widehat{\mathbf{cl}_{mo}}(\bar{d}, \bar{t})$ denotes the vector of predicted responses, n is the sample size, and p is the number of parameters (equal to two in this case).

Parameter estimation is performed using the Levenberg-Marquardt method [118], which minimises the sum of squared errors and provides the parameter covariance matrix:

$$\sum_p = \sigma_\varepsilon^2 [\mathbf{J}^T \mathbf{J}]^{-1} \quad (42)$$

where Σ_p is the covariance matrix of the parameters, $\mathbf{J}$ is the Jacobian matrix of the model function evaluated at the estimated parameter values.

To retain the time dependency of the diffusion process, the data-driven apparent diffusion coefficient is converted into a data-driven chloride diffusion coefficient $D_{RCM,0,data}$ following the formulation provided in [67]:

$$D_{RCM,0,data} = \frac{D_{app}(\bar{t})}{k_e k_t A(\bar{t})} \quad (43)$$

where k_e , k_t , and $A(\bar{t})$ are the environmental regression parameter, the transfer parameter, and the ageing function, respectively [67]. This ensures that the updated model accounts for both environmental effects and concrete ageing.

Accordingly, Eq. (39) can be reformulated when no initial chloride ions are present:

$$cl_{mo,data}(d, t) = C_{s,app,data} \left[1 - \operatorname{erf} \left(\frac{d - \Delta d}{2\sqrt{k_e D_{RCM,0,data} k_t A(\bar{t}) \cdot t}} \right)^2 \right] \quad (44)$$

The data-driven model described by Eq. (44) can be used to evaluate chloride concentration at the cover depth for a given year. The data-driven parameters $C_{s,app,data}$ and $D_{app,c}(\bar{t})$ can be associated with PDFs. These PDFs are used as the likelihood functions of $C_{s,app,data}$ and $D_{app,c}(\bar{t})$ because they incorporate the data evidence and the relative uncertainties. Following from the fitting procedure, the two PDFs can be defined as:

$$D_{app,c}(\bar{t}) \rightarrow L_{D_{app,c}(\bar{t})}(D_a) \sim N(D_a, \mu_D; \sigma_D) \quad (45)$$

$$C_{s,app,data} \rightarrow L_{C_{s,app}}(C_l) \sim N(C_l, \mu_C; \sigma_C) \quad (46)$$

where μ_C and μ_D represent the expected value of $C_{s,app,data}$ and $D_{app,c}(\bar{t})$, respectively, while σ_C and σ_D are estimated through Eq. (41).

Eq. (43) provides the likelihood for $D_{RCM,0,data}$, denoted as $L_{D_{RCM,0}}(D_0)$. This function does not follow a parametric distribution and must be evaluated numerically or by simulation. Using Eq. (14), the updated posterior distributions $p''_{C_{s,app}}(C_l)$ and $p''_{D_{RCM,0}}(D_0)$ are obtained. Finally, the predictive distribution of the time to pitting corrosion is derived using Eq. (15).

In this formulation, the prior distribution of T_p is defined by Eq. (38), with all the parameters ($K_c, k_e, A(\bar{t}), C_{cr}, D_{RCM,0}, C_{s,app}$) treated probabilistically via their prior distributions. However, since only $D_{RCM,0}$ and $C_{s,app}$ are updated, the prior of f'_{T_p} is expressed conditionally as $f'_{T_p}(K_c, k_e, A(\bar{t}), C_{cr} | D_{RCM,0}, C_{s,app})$, allowing the use of Eq. (15).

7.2 Propagation phase

The realisation of the corrosion activation event ($\mathbb{C}_i$) marks the onset of the propagation phase, during which the steel cross-section, ductility, and tensile strength progressively decrease. In this work, the propagation phase is modelled as a loss of steel cross-section in the reinforcement bars, which is the dominant factor when corroded elements are assumed to fail upon reaching their yield stress, and as both a reduction of cross-section and degradation of mechanical properties in the prestressing wires (see also Section 7.3).

For reinforcing bars, the corrosion rate, V_{corr} , is defined following [113]. In addition to the influence of relative humidity, the effect of temperature on the corrosion rate is incorporated. The temperature dependency is modelled using the Arrhenius law, based on the mean annual temperature at the site, as recommended in the Model Code for Service Life Design [66]. This formulation allows the corrosion rate to vary consistently with temperature changes over time:

$$V_{corr} = \exp\left(b_e \left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right) V_{corr,a} ToW \quad (47)$$

where b_e is the activation energy [K], T_{ref} and T_{real} are the reference temperature equal to 293 K, and the mean annual temperature [K], $V_{corr,a}$ is the active corrosion rate [mm/year] and ToW is the time of wetness expressed as an annual fraction. Assuming a constant corrosion rate, the annual reduction in bar diameter, $r(t)$, can be estimated as:

$$r(t) = V_{corr} \alpha (t - T_{g|p}) \text{ given } \mathbb{C}_i \quad (48)$$

where α is the pitting depth factor [-], equal to 2 for generalised corrosion or modelled as $\alpha \sim LN(9.28, 4.04)$ [67] for pitting corrosion; and $T_{g|p}$ denotes the

time to initiation of generalised or pitting corrosion, expressed in years. The residual area, $A_s(t)$, is then given by Eq. (49):

$$A_s(t) = \frac{\pi(D_0 - r(t))^2}{4} \quad (49)$$

where D_0 [mm] is the initial diameter of the stirrup or longitudinal reinforcement. This equation can be adapted for consistency with the general Eq. (8) presented in Section 4.3:

$$\begin{aligned} A_{s,i}(t) &= A_{s,i}(0) - I[t - T_{g|p,i}]H_i(\boldsymbol{\eta}, t) \\ H_i(\boldsymbol{\eta}, t) &= \frac{\pi}{4} \left(r_i^2(\boldsymbol{\eta}, t) - 2D_0r_i(\boldsymbol{\eta}, t) \right) \end{aligned} \quad (50)$$

where the $\boldsymbol{\eta}$ vector contains all the variables presented in Eqs. (47) and (48), and the equation is explicitly referred to section i , making it possible to account for spatial dependencies.

Moreover, in the case of generalised corrosion, spalling is also included. The following LSF models the spalling event:

$$g_{sp}(t) = \frac{A_{s,i}(0) - A_{s,i}(t)}{A_{s,i}(0)} - 0.1 = \frac{I[t - T_{g,i}]H_i(\boldsymbol{\eta}, t)}{A_{s,i}(0)} - 0.1 \quad (51)$$

Spalling is considered to occur when $g_{sp}(t) > 0$. The underlying assumption is that spalling initiates once the corrosion-induced loss of reinforcement cross-sectional area reaches 10%, as proposed in [119].

To evaluate the effective web width after spalling $b_{w,eff}$, the model proposed in [120] is implemented:

$$b_{w,eff} = \begin{cases} b_w - 2(c_w + \phi_w) + \frac{u_w}{5.5} & \text{if } u_w \leq 5.5(c_w + \phi_w) \\ b_w - \frac{5.5}{u_w}(c_w + \phi_w)^2 & \text{if } u_w > 5.5(c_w + \phi_w) \end{cases} \quad (52)$$

where b_w is the web width of the section before spalling, c_w is the stirrups' concrete cover, ϕ_w is the stirrup diameter, and u_w is the stirrup spacing.

The effect of corrosion on mechanical characteristics of the reinforcements is also modelled. The model proposed by Yu et al. [87] is adopted:

$$\begin{aligned} \varepsilon_{syc} &= \frac{f_{syc}}{E_s} \\ f_{syc} &= \frac{1 - 1.092\xi_s}{1 - \xi_s} f_{sy} \\ \varepsilon_{suc} &= \varepsilon_{su} \cdot \exp(-2.556\xi_s) \end{aligned} \quad (53)$$

where $E_s = 210 \text{ GPa}$ is the steel Young's modulus, f_{sy} is the yield stress of the steel, ε_{su} is the ultimate strain of the steel, ξ_s is the corrosion degree defined as the ratio of the steel lost area to the intact steel area, f_{syc} is the yielding stress of the steel at a specific corrosion degree, ε_{su} is the ultimate strain of the steel at a specific corrosion degree, and ε_{syc} is the yielding strain of the steel at a specific corrosion degree.

The same formulations used to estimate the residual cross-sectional area are applied to prestressing wires subjected to generalised corrosion, whereas dedicated models are adopted for pitting corrosion, given its highly localised and more severe impact.

In particular, to quantify the evolution of pitting corrosion in wires and to evaluate the maximum pit depth $a(t)$, the probabilistic model proposed by Darmawan and Stewart [121] is employed. This model was derived from laboratory tests on prestressing wires and assumes that the maximum pit depth follows a Gumbel distribution. Despite the development of pit depth can be interpreted as occurring in stages [122], the assumption of a (type I) extreme value distribution is widely supported in the literature [123], [124], [125], [126], [72].

The probability density function $f_a(t, i_c, l)$ of the maximum pit depth over time $a(t)$ is:

$$f_a(t, i_c, l) = \frac{\alpha_s}{\lambda^{0.54}} \exp\left(-\alpha_s \left(\frac{a}{\lambda^{0.54}} - \mu\right)\right) \exp\left(-\exp\left(-\alpha_s \left(\frac{a}{\lambda^{0.54}} - \mu\right)\right)\right); \quad (54)$$

$$t > T_p$$

In Eq. (54) i_c is the corrosion current density [mA/cm^2], l is the wire length considered subjected to corrosion [mm], and $\alpha_s = 8.10$ [-] is the scale parameter. The remaining parameters can be obtained as:

$$\lambda = \frac{\left[d_0^2 - \left(d_0 - 0.0232i_c \cdot (t - T_p)\right)^2\right]}{[d_0^2 - (d_0 - 0.0232i_c T_0)^2]} \quad (55)$$

$$T_0 = \frac{i_{c-exp} T_{0-exp}}{i_c} \quad (56)$$

$$\mu = \mu_{0-exp} + \frac{1}{\alpha_s} \ln\left(\frac{l}{l_{0-exp}}\right) \quad (57)$$

In the previous equations, d_0 is the wire's diameter [mm], $i_{c-exp} = 186$ is the test electric current density [mA/cm^2], $T_{0-exp} = 0.3836$ is the test time of exposure [years], $\mu_{0-exp} = 0.84$ is the test location parameter, and $l_{0-exp} = 650$ is the wires' length used during the laboratory test [mm] [121].

Based on experimental results reported in [84], [85], pitting corrosion in prestressing wires may develop in three geometric shapes: hemispherical, concave, and planar. However, [85] showed that the concave shape occurs only at high corrosion levels and in rare cases, whereas hemispherical and planar morphologies are observed with similar probability.

Therefore, two models (see Figure 7-2) are employed to evaluate wire area loss, with each wire randomly assigned to one of the two morphologies with equal probability (50%). The loss of cross-sectional area of a wire for a given pit depth can be calculated using the following expressions:

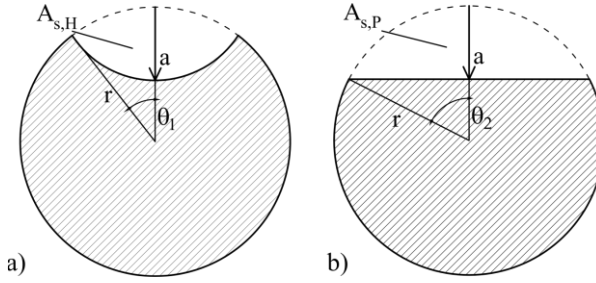


Figure 7-2: Considered corrosion morphologies: a) hemispherical pit, b) planar pit.

Hemispherical case

$$A_{s,h}(t) = 2r_0^2(\theta_1 - \sin \theta_1 \cos \theta_1)$$

$$\theta_1 = \arccos\left(1 - \frac{a(t)}{2r_0}\right)$$

Planar case

$$A_{s,p}(t) = r_0^2(\theta_2 - \sin \theta_2 \cos \theta_2)$$

$$\theta_2 = \arccos\left(1 - \frac{a(t)}{r_0}\right)$$

(58)

In Eq. (58), r_0 is the wire radius, $a(t)$ ranges between 0 and $2r_0$, and the angles θ_1 and θ_2 are defined in Figure 7-2. Eq. (58) can be adapted to be consistent with the general framework and to explicitly account for spatial dependency, as provided for Eq. (49) through Eq. (50).

$$A_{s,i}(t) = A_{s,i}(0) - I[t - T_{p,i}]H_i(\boldsymbol{\eta}, t)$$

$$H_i(\boldsymbol{\eta}, t) = A_{s,p_i}(t) \text{ or } A_{s,h_i}(t)$$

(59)

where $H_i(\boldsymbol{\eta}, t)$ is evaluated through Eq. (58) depending on the realised corrosion morphology.

The models for reinforcements and wires, defined for both generalised and pitting corrosion cases, can be used to estimate the progressive reduction in cross-section

over time. In this way, they provide the time-dependent degradation functions required by the limit state formulations and ensure that corrosion progression is coherently integrated into the system reliability analysis framework.

7.3 Specificities for Prestressed bridges with post-tensioned tendons

As discussed in Chapter 5, injection voids compromise the protective function of grout. In such cases, wires are directly exposed to aggressive agents, leading to harsher environmental conditions than in fully grouted ducts. Accordingly, two models for the critical chloride threshold are associated with these conditions: C_{cr_c} representing the worst-case condition of reduced protection in the presence of voids [55], [58], [78]; C_{cr_s} representing the more favourable case where grout fully encloses the wires. Both models should remain consistent with chemical and damage analyses of tendons observed in existing structures, as reported, for example, in [78].

7.3.1 Tensile strength of a tendon subjected to degradation

A tendon is composed of a bundle of prestressing wires, all subjected to the same initial force. As corrosion reduces wire cross-sections, the sudden rupture of one wire triggers the redistribution of the applied force to the remaining ones. This redistribution may lead to cascading failures and, ultimately, tendon rupture, even in the absence of external loads. To capture this behaviour, a dedicated model for the tensile capacity (R_{tot}) of a corroding tendon is introduced.

The model distinguishes between sound and corroding wires. Sound wires fail in a ductile manner when their tensile strength is reached, while corroding wires fail in a brittle manner once their maximum stress capacity ($\sigma_{w,max}$) is exceeded. The redistribution of the force sustained by a broken wire to the other wires in a tendon is modelled through a brittle Daniel System. This representation allows the modelling of the progressive loss of load-carrying capacity of the tendon under limited post-failure redistribution. The model becomes more appropriate as corrosion advances due to the reduction in ductility. Moreover, tendon resistance evaluated using Eq. (60) is compared with the residual prestressing force applied to the tendon (P_{tot}), and the tendon is excluded from the flexural capacity evaluation when the prestressing force exceeds the tendon resistance.

$$R_{tot}(t) = f_{pu} \cdot (n_w - n_c) \cdot A_s(0) + \max_j \{ (n_c - j + 1) A_{s,j}(t) \sigma_{w,max,j}(t) \} \text{ given} \quad (60)$$

$$P_{tot} < R_{tot}(t)$$

Here, f_{pu} is the ultimate stress, n_c the total number of corroding wires depending on the development of the corrosion process, $A_{s,j}(t)$ the remaining areas of wires subjected to corrosion (with $j = 1, \dots, n_c$) sorted in ascending order, n_w the total number of wires in a cable, $A_s(0)$ is the area of a wire a time equal to zero, and $\sigma_{w,max,j}(t)$ is the maximum stress capacity of the wires as a function of pitting depth, evaluated using the model proposed (specifically developed for wires) by Yu et al. [87], through Eq. (61) depending on the corrosion degree $\mu_{loss}(t)$.

In particular, when the corrosion degree $\mu_{loss}(t)$ is smaller than the critical level μ_{lim} , the stress-strain relationship of a corroded wire can be modelled as follows (the explicit dependency on time t is omitted here for simplicity of notation):

$$\sigma_{w,max} = \begin{cases} \varepsilon_{pu,cor} E_p & ; \varepsilon_{pu,cor} \leq \varepsilon_{p0} \\ 0.85 f_{pu,cor} + (\varepsilon_{pu,cor} - \varepsilon_{p0}) \left(\frac{0.15 f_{pu,cor}}{\varepsilon_{pu,cor} - \varepsilon_{p0}} \right) & ; \varepsilon_{p0} < \varepsilon_{pu,cor} \end{cases} \quad (61)$$

where $E_p = 210 \text{ GPa}$ is the wire Young's modulus, $f_{pu,cor}$ is the ultimate stress of a corroded wire, $\varepsilon_{pu,cor}$ is the ultimate strain of a corroded wire, and $\varepsilon_{p0} = \frac{0.85 f_{pu}}{E_p}$ is the yielding strain of an uncorroded wire, with f_{pu} being the ultimate stress of an uncorroded wire.

Instead, when $\mu_{loss}(t) \geq \mu_{lim}$, the stress-strain relationship of a corroded wire is:

$$\sigma_{w,max} = \varepsilon_{pu,cor} E_p \quad (62)$$

The ultimate stress of a corroding wire can be evaluated as follows [87]:

$$\begin{aligned} f_{pu,cor}(t) &= f_{pu} \cdot \frac{\alpha_{pu}}{1 - \mu_{loss}(t)} \\ \alpha_{pu}(t) &= 1 - 1.935 \mu_{loss}(t) \end{aligned} \quad (63)$$

The ultimate strain of a corroded wire can be estimated as:

$$\begin{aligned} \varepsilon_{pu,cor}(t) &= (1 - 9.387 \mu_{loss}) \varepsilon_{pu} & ; \mu_{loss}(t) < \mu_{lim} \\ \varepsilon_{pu,cor}(t) &= \frac{f_{pu,cor}}{E_p} & ; \mu_{loss}(t) \geq \mu_{lim} \end{aligned} \quad (64)$$

where ε_{pu} is the ultimate strain of an uncorroded wire.

Finally, μ_{loss} is defined as the ratio of the wire lost area to the intact steel area and depend on the maximum pit depth $a(t)$ (Eq. (54)). μ_{lim} is the limit value of the cross-sectional loss for which the hardening phase disappears. Yu et al. [87] propose μ_{lim} is fixed at 8%, however, to avoid discontinuities in the stress-strain

relationship, μ_{lim} is here estimated by imposing that, when $\mu_{loss} = \mu_{lim} = \bar{\mu}$, the two formulations in Eq. (64) must produce the same $\varepsilon_{pu,cor}$:

$$\frac{\frac{\alpha_{pu}}{1 - \bar{\mu}} f_{pu,cor}}{E_p} = (1 - 9.387\bar{\mu}) \varepsilon_{pu} \quad (65)$$

The solution of the equality in Eq. (65) leads to:

$$\bar{\mu} = \frac{10.387K - 1.935 - \sqrt{(10.387K - 1.935)^2 - 4(K - 1) \cdot 9.387K}}{18.774K} \quad (66)$$

$$K = \frac{\varepsilon_{pu} E_p}{f_{pu}}$$

Using Eq. (66), continuity of the stress-strain relationship is ensured while also accounting for uncertainty in the value of μ_{lim} .

7.4 Summary of the Chapter

This chapter presents the modelling of corrosion degradation processes for both reinforcing bars and prestressing wires. The activation phase is described by logical systems that combine carbonation and chloride ingress, while probabilistic models of section loss and pit growth represent the propagation phase. For prestressed tendons, the specific effects of injection voids and the coexistence of ductile and brittle wire behaviour are incorporated through a partially ductile Daniels system model.

Together, these models provide the time-dependent degradation functions required by limit-state formulations and ensure that the physical processes of corrosion are coherently integrated into the system reliability framework.

In the following chapter, the proposed framework is applied to case studies, demonstrating its ability to calculate the structural reliability of existing concrete bridges and estimate their residual service life under progressive degradation.

8 Application of the Framework to Case Studies

The developed system reliability framework (Chapter 4), together with the probabilistic model for tendon degradation (Chapter 5), the spatial reliability approach (Chapter 6), and the corrosion degradation model (Chapter 7), is applied to two case studies. The aim is to demonstrate the potential of the framework in estimating bridge service life and to illustrate its practical applicability. The case studies and results build on the applications described in the appended papers but also provide new analyses and more comprehensive outcomes.

8.1 Post-tensioned bridge

A typical PT bridge found on the Italian highway network consists of simply supported decks, each composed of several parallel girders. A simply supported deck, part of a bridge structure built in 1976, with four parallel girders, is considered to illustrate the proposed system reliability framework. The deck has a span length of 42 meters, a slab width of 12.3 meters, and a total section height of 2.6 meters, as shown in Figure 8-1. Each girder is post-tensioned with six cables, each containing 42 parallel wires. The shear reinforcement varies along the girder length and is symmetrical with respect to the midspan. It consists of stirrups of diameter $\phi 12$ at 15 centimetres spacing from the support to 2.15 metres, followed by $\phi 10$ at 25 centimetres spacing over the next 10 metres, and $\phi 8$ at 25 centimetres spacing from 12.15 metres from the support to the midspan (see Figure 8-1d)

Following the reliability framework, two main failure modes are considered: bending and shear. The deck is represented as a nested system (see Figure 8-2), where each analysed section acts as a component of a girder series system, and all girders together form the deck system. The overall deck configuration depends on the considered consequence scenario. Moreover, the approximation introduced by employing the DS modelling for the evaluation of global capacity is quantified through comparison with a NLFE model of the deck under two representative load configurations and four corrosion levels applied to the external girder.

multiple girders fail concurrently, total bridge collapse may occur, and the deck system is then modelled with sections of different girders as components within a ductile or brittle Daniels Systems [19], [22]. Generally, all possible combinations of failing sections are considered, resulting in several DSs arranged in series (see Figure 8-2c). For shear failure or corrosion-related scenarios, a brittle Daniels System is usually the most appropriate model (see, for example, [81]).

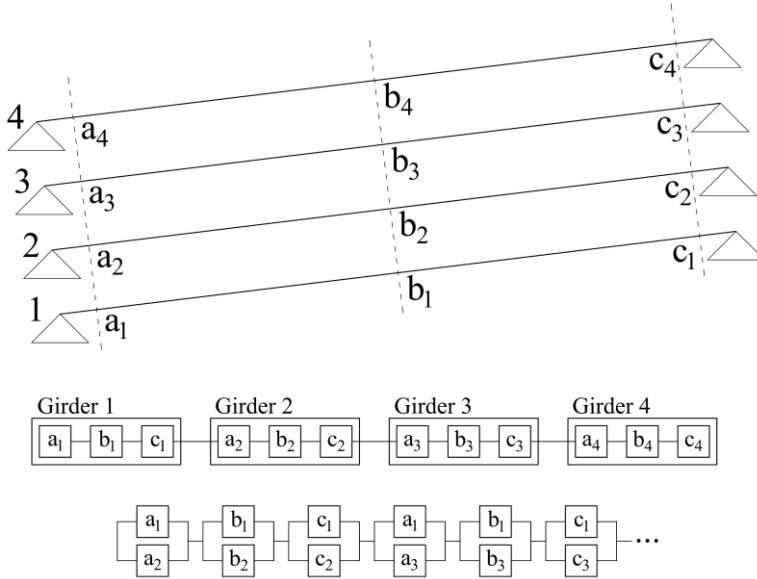


Figure 8-2: a) Schematic representation of the bridge deck, b) series system configuration, and c) series of brittle Daniels Systems.

8.1.1 Nonlinear Finite Element model of the PT deck

A NLFE model of the considered PT bridge deck is developed using the SAP2000 software. Girders and crossbeams are modelled using beam elements, while the slab is modelled using shell elements (Figure 8-3). The slab is included primarily for load introduction, while its stiffness is reduced to avoid double-counting its contribution to the structural capacity, as the main girders and crossbeams T-shaped sections already include the slab contribution.

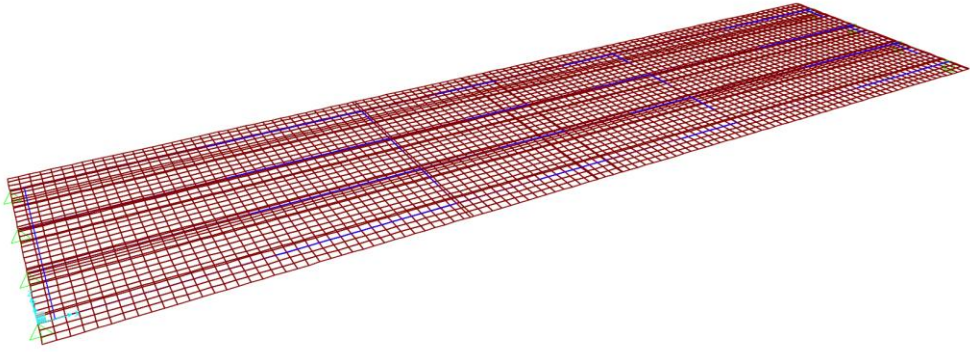


Figure 8-3: Developed grillage model of the PT deck

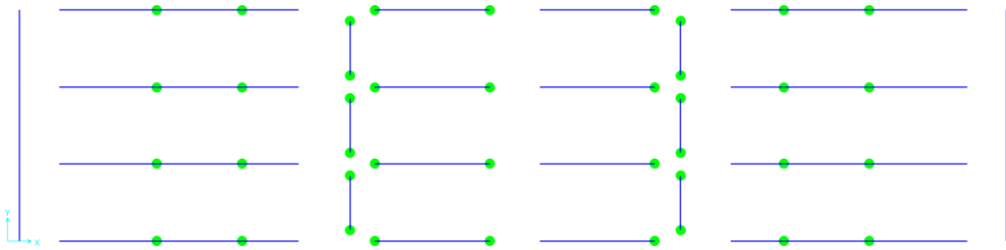


Figure 8-4: Positions of the modelled plastic hinges

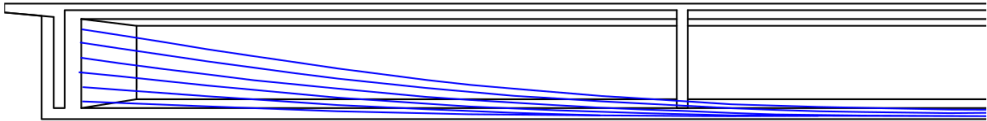


Figure 8-5: Representation of the tendons layout.

Nonlinearity is modelled through plastic hinges placed at the ends of the beam elements, at midspan, at the locations where crossbeams intersect the main girders, and at 6 m and 11 m from the girder ends (see Figure 8-4 and Figure 8-5). The plastic hinges are defined considering the mean values of both the concrete compressive strength and the steel yield strength. The use of mean values is justified, as the plastic hinge is intended to represent a typical section resistance rather than the characteristic or design resistance values. Moreover, plastic hinges are defined for four corrosion cases: no corrosion, and cables area reductions of 5%, 10%, and 15%. To account for the reduced ductility of the plastic hinge in the presence of corrosion, the mechanical properties of the wires, as a function of the

corrosion level, are modelled following the results provided by Yu et al. [87] as introduced in Section 7.3.1.

Figure 8-6 shows the Moment-Curvature diagrams for the values of $\mu_{loss} = [0, 5\%, 10\%, 15\%]$ for the midspan section of the external girder.

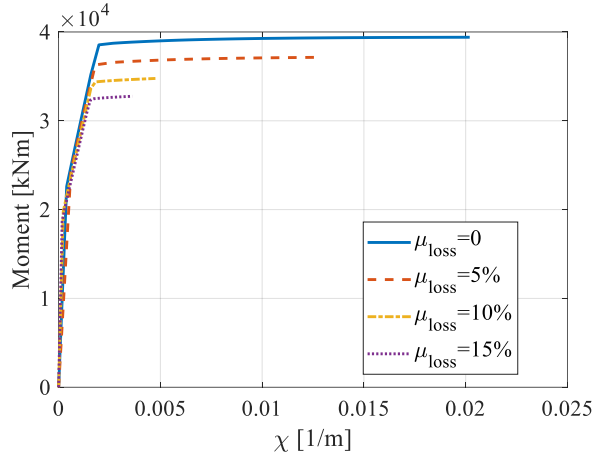


Figure 8-6: Moment-Curvature diagrams for the midspan section of the external girder at different corrosion levels

To obtain the plastic rotation, the hinge length is modelled as equal to the depth of the beam section [91].

8.1.1.1 Load configuration

In order to investigate the effect of the load position on the global capacity, two cases are considered: (i) the tandem load is placed at the midspan section, and the distributed load is applied over the entire carriageway; (ii) the tandem load is placed 2.6 metres from the support, i.e. a distance equal to the height of the main girder, and the distributed load is applied to the carriageway beyond the initial 2.6-metre region. The geometry of the load scheme corresponds to Load Model 1 of Eurocode 1 [127], including the load magnitudes. Both load cases are illustrated in Figure 8-7.

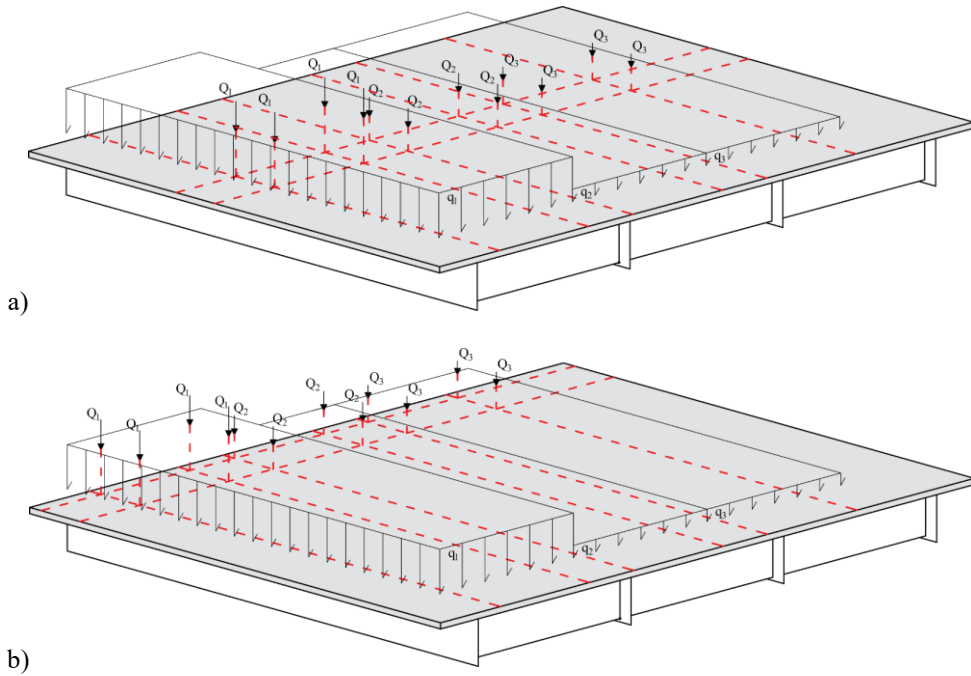


Figure 8-7: Schematic representation of the considered load configurations on the deck: a) the tandem load located at midspan; b) the tandem load located 1.2 metres from the support.

8.1.1.2 Outcomes of the NLFE analysis

A nonlinear static (push-down) analysis is performed in force control. In particular, the load intensity is gradually increased until convergence can no longer be achieved, i.e. until the stiffness matrix becomes singular.

A total of eight push-down analyses are carried out, four for each load configuration, by varying the corrosion degree of one of the external girders. In particular, the hinges applied to the most heavily loaded external girder (Figure 8-7) are modified according to the moment–curvature diagrams corresponding to different corrosion degrees. The results of the analyses are presented in Figure 8-8 in terms of load–displacement curves. The plotted displacement corresponds to the vertical displacement at the midspan of the most heavily loaded external girder. The maximum load multiplier (λ_{FEM}) represents the global capacity of the deck; a load multiplier equal to 1 corresponds to the characteristic load as defined in [127].

From the graphs, the global load multiplier ranges between 2.97 and 8.54 depending on the load configuration and the corrosion degree. Table 8-1 summarises the estimated values.

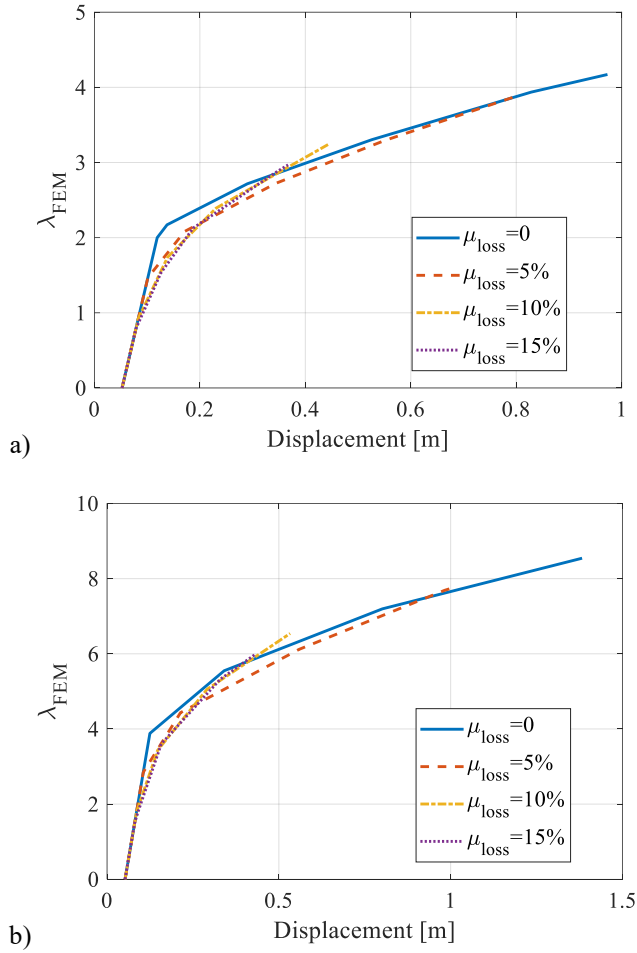


Figure 8-8: Load multiplier evaluated using the NLFE model for: a) tandem load placed at midspan; b) tandem load placed near the support.

Table 8-1: Estimated load multipliers for both load cases at varying corrosion levels obtained using the NLFE model

Corrosion degree (μ_{loss})	λ_{FEM} for tandem load at the midspan	λ_{FEM} for tandem load in proximity of the support
0%	4.17	8.54
5%	3.90	7.79
10%	3.25	6.54
15%	2.97	5.99

8.1.1.3 Comparison between the NLFE and the adopted brittle Daniels System estimated load multipliers

The load multipliers for the two load cases are evaluated using the brittle DS modelling proposed in the thesis. In particular, for both load configurations, the moment and shear demands are evaluated for the three main girders using the Courbon method, and for each section the ratio of capacity to demand is calculated. In this case, both capacity and demand for each section i are evaluated using the proposed brittle DS formulation:

$$C_{S,i} = \max\{2 \cdot \min[C_{1,i}, C_{2,i}], \max[C_{1,i}, C_{2,i}]\} \quad (67)$$

$$D_{S,i} = D_{1,i} + D_{2,i}$$

where $C_{S,i}$ is the system capacity, $D_{S,i}$ is the system demand due only to the traffic loads, $C_{1,i}$ is the capacity of the external girder, $C_{2,i}$ is the capacity of the internal girder, and $D_{1,i}$ and $D_{2,i}$ are the respective demands due only to the traffic loads. All these quantities are evaluated for the section i , and the load multiplier is defined as the minimum, over all sections, of the ratio between the available traffic load-carrying capacity, i.e. the system capacity reduced by the permanent load effects $G_{1,i}$ and $G_{2,i}$, and the traffic load demand:

$$\lambda_{bDS} = \min_i \left\{ \frac{C_{S,i} - G_{1,i} - G_{2,i}}{D_{S,i}} \right\} \quad (68)$$

Table 8-2 shows the calculated load multipliers using the proposed brittle DS modelling.

Table 8-2: Estimated load multiplier for both load cases at varying corrosion degrees using the bDS modelling

Corrosion degree (μ_{loss})	λ_{bDS} for tandem load at the midspan	λ_{bDS} for tandem load in proximity of the support
0%	3.50	7.21
5%	3.20	6.61
10%	2.92	6.01
15%	2.66	5.48

To compare the load multipliers evaluated using the NLFE and the bDS models, the relative error is computed for both load cases. The resulting values of ε_S are depicted in Figure 8-9:

$$\varepsilon_S = \frac{\lambda_{bDS} - \lambda_{FEM}}{\lambda_{FEM}} \cdot 100 \quad (69)$$

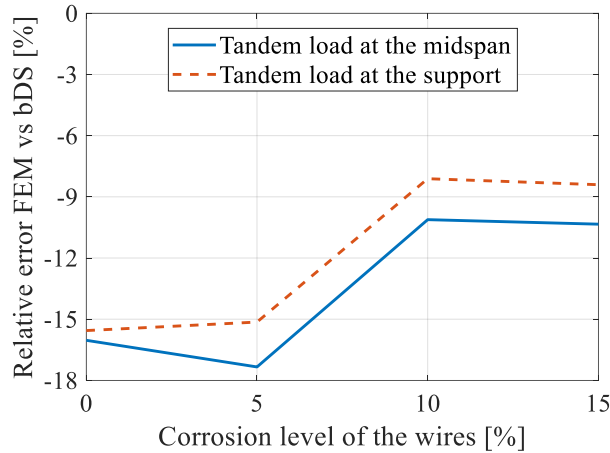


Figure 8-9: Relative error of the load multipliers evaluated using the brittle Daniels System model with respect to the NLFE model.

Figure 8-9 shows that, as expected, the simplified representation of the system behaviour through a brittle Daniels System is conservative, as the resulting load multipliers are smaller than those estimated using the NLFE model. For the first load case, where the tandem load is placed at midspan, the error ranges between -18% and -10% , depending on the corrosion level of the external girder. For the second load case, the error ranges between -16% and -8% . Moreover, the error is significantly influenced by the considered load configuration, confirming that the degree of conservativeness depends on the underlying redistribution mechanisms. Overall, the level of conservativeness can be considered moderate, indicating that the proposed DS modelling provides an approximate yet sufficiently accurate estimate of the system capacity.

Furthermore, these results underline that the plastic redistribution capacity of the deck depends not only on the deck geometry and tendons layout but also on the load configuration. Consequently, when spatial variability involving multiple potential failure modes and critical sections is considered, nonlinear probabilistic analyses would, need to be performed and the load should be represented as a time dependent spatial random field. Such a computational burden is not compatible with a probabilistic framework aimed at representing degradation across several sections and elements over time, particularly within a time-variant reliability context, thus motivating an approximate analytical closed-form method for the estimation of the system capacity, such as the one proposed herein.

This limitation is also acknowledged in the literature (e.g., [89], [95]), where, when nonlinear probabilistic FE models are employed to estimate system capacity, only one load configuration and one critical failure section are typically considered. This

section is usually the midspan section subjected to bending failure, with traffic loads placed at midspan, as it is generally identified as the dominant failure mode.

However, when spatial spreading of the degradation is accounted for, the most utilised sections alone do not dominate the system reliability (see also Section 8.1.4), as also less utilised section, but with higher level of degradation have comparable failure probability. Therefore, the most critical loading position cannot be clearly determined, and multiple loading conditions must be considered to provide a relevant estimation of the system reliability.

8.1.2 Limit State Functions

The equations provided in Chapters 4 and 6 can be adapted to both the bending and the shear failure modes. In particular, Eq. (7) can be reformulated for both modes, while Eq. (29) applies specifically to bending, yielding the following expressions:

$$g_{M,i,k}(\mathbf{X}, \mathbf{S}, t, k, \varsigma_{i,k}) = \theta_{M,C} M_C(\mathbf{X}, t, k, \varsigma_{i,k}) - \theta_{M,D} M_D(\mathbf{S}, k, \varsigma_{i,k}) \quad (70)$$

$$g_{V,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{V,C} V_{C,i}(\mathbf{X}, t) - \theta_{V,D,i} (V_{D,i}(\mathbf{S}) - V_{P,i}(P_{tot})) \quad (71)$$

In Eq. (70), $\theta_{M,C}$ represents the model uncertainties for the bending capacity model, $M_C(\mathbf{X}, t, k, \varsigma_{i,k})$ denotes the bending capacity as a function of the capacity variables $\mathbf{X}$, time t , simulation k at section $\varsigma_{i,k}$. $\theta_{M,D}$ represents model uncertainties for the bending demand model, and $M_D(\mathbf{S}, k, \varsigma_{i,k})$ gives the bending demand depending on the vector of demand variables $\mathbf{S}$ and modelled here as time-independent. In Eq. (71), $\theta_{V,C}$ represents the model uncertainties for the shear capacity model, $V_{C,i}(\mathbf{X}, t)$ represents the shear capacity at the time t and section i . The shear demand model includes $\theta_{V,D}$ for model uncertainties and $V_{D,i}(\mathbf{S})$ for shear demand at section i . $V_{P,i}(P_{tot})$ is the vertical component of the prestressing force P_{tot} at section i . Notably, shear-related variables do not depend on the simulation index; thus, sections are denoted as i , as they remain the same across all simulations k .

Finally, Eq. (30) can be directly applied to evaluate the total failure probability by defining the event $F(t)$ as the union of the failure events related to bending and shear failure modes.

8.1.2.1 Bending Capacity

The bending capacity model for prestressed concrete is based on the strength of the prestressing steel, as provided by code procedures [128]. When the neutral axis cuts the section at the slab level, the following equation is used:

$$M_C(\mathbf{X}, t, k, \varsigma_{i,k}) = \sum_{l=1}^{n_p} (R_{tot,l}(t) \cdot d_l(\varsigma_{i,k})) + 0.32 f_c(\varsigma_{i,k}) b(\varsigma_{i,k}) y_c^2(\varsigma_{i,k}) \quad (72)$$

where n_p is the total number of cables, $R_{tot,l}(t)$ is the yield resistance of the l -th cable at time t evaluated according to Eq. (60), $d_l(\zeta_{i,k})$ is the inner lever arm of the l -th cable evaluated from the neutral axis accounting for the relative geometric uncertainties (θ_d) based on [99], $f_c(\zeta_{i,k})$ is the concrete compressive strength, $b(\zeta_{i,k})$ is the slab section width accounting for shear lag, and $y_c(\zeta_{i,k})$ is the position of the neutral axis. All variables are evaluated at section $\zeta_{i,k}$.

This formulation provides accurate results for undamaged prestressed concrete girders. In the damaged case, however, the bending capacity at each section is determined iteratively by balancing tensile and compressive forces in the section, estimated using modified stress-strain relationships for the corroded steel and a parabola-rectangle stress-strain diagram for the concrete. In this case, Eq. (72) is employed as the initial estimate within the iterative procedure.

8.1.2.2 Shear Capacity

The shear capacity model is based on the modified compression field theory, the current standard design model in Eurocode 2 [128]. For sections near the supports ($s_i \leq z_i \cot \theta_i$), Eq. 8.55 in [128] applies:

$$V_{C,i}(\mathbf{X}, t) = \min \left(\begin{array}{c} 0.5f_{c,i}b_{w,eff,i} \frac{z_i \cot \theta_i - s_i}{1 + \cot^2 \theta_i} + \sum_{l=1}^{n_{st}} A_{sw,l}(t) \cdot f_{yw,i} \\ 0.5f_{c,i} \frac{\cot \theta_i}{1 + \cot^2 \theta_i} z_i b_{w,eff,i} \end{array} \right) \quad (73)$$

where $f_{c,i}$ is the concrete resistance, z_i is the inner lever arm of the stirrups, $b_{w,eff,i}$ is the section web width accounting for the spalling, θ_i is the angle of the compressive strut against the beam longitudinal axis, s_i is the distance from the support of the considered section, n_{st} is the number of stirrup intersected by the strut, $A_{sw,l}(t)$ is the area of each stirrup at time t , and $f_{yw,i}$ is the stirrup yielding stress. All the variables are evaluated at the section i and Eq. (73) can be adjusted to account for the reduction in the stirrup area due to corrosion. When $s_i > z_i \cot \theta_i$ the concrete strut contribution is set to zero, as it no longer contributes to shear capacity.

Eqs. from (70) to (73) are incorporated in Eqs. (1), (2), and (3) to define the LSFs for girders and the deck, considering both major repair and global collapse scenarios. Eq. (30) is then used to evaluate the total failure probability and the associated reliability index.

8.1.2.3 Limit State Functions for the Crossbeams

Given the bDS approximation utilised to estimate the system capacity, an elastic transverse load distribution is used to define the transverse distribution of traffic effects among girders for a given load position. Therefore, the reliability of the crossbeams needs to be evaluated to verify whether the crossbeams are able to transfer the load throughout the whole timespan analysed. The progressive

redistribution associated with member yielding and loss of capacity is instead represented through the Daniels load-sharing mechanism in the system resistance model, which is estimated moderately conservative through the push-down analyses provided (Section 8.1.1.3).

To estimate the action on the crossbeams, the Courbon method is again employed. A section located at a distance equal to the crossbeam height from the external girder is assessed with respect to shear in the reliability analysis. The following time dependent LSF is defined, and Figure 8-10 illustrates the detailed reinforcement layout of the crossbeams:

$$g_{CB,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{V,C} V_{C,CB,i}(\mathbf{X}, t) - \theta_{V,D} V_{D,CB,i}(\mathbf{S}) \quad (74)$$

where $V_{C,CB,i}(\mathbf{X}, t)$ is the shear capacity of the cross-beam as a function of the vector of capacity variables $\mathbf{X}$ and time t , $\theta_{V,C}$ and $\theta_{V,D}$ represent the model uncertainties for the shear capacity and demand models, and $V_{D,CB,i}(\mathbf{S})$ is the shear demand evaluated using the Courbon method depending on the vector of demand variables $\mathbf{S}$. Both $V_{C,CB,i}(\mathbf{X}, t)$ and $V_{D,CB,i}(\mathbf{X}, t)$ are referred to the i -th section analysed

The shear capacity can be evaluated using Eq. (73) as adopted for the main girders.

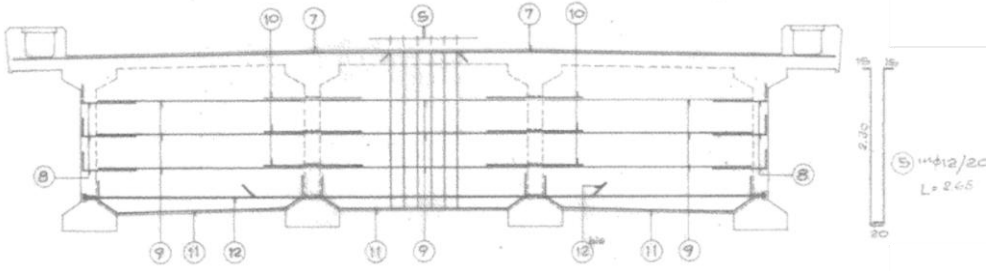


Figure 8-10: Excerpt of the original documentation showing the reinforcements details of the crossbeams

8.1.2.4 Load modelling

The permanent load per girder is estimated at 55 kN/m, while traffic loads are calculated using Load Model 1 from Eurocode 1 [127]. Load effects are determined using Euler–Bernoulli beam theory, with the Courbon method applied to distribute loads transversely among girders (see Figure 8-11). In accordance with Eurocode 1 [127], the notional lanes and remaining areas are arranged to produce the most adverse effect under consideration. Since loading the remaining area would reduce the demand on the external girder (girder 1 in Figure 8-11), it is not loaded in this configuration.

The traffic load statistics used in the yearly reliability assessment are derived from the Eurocode 1 background [129] formulation. Although the characteristic load corresponds to a 5% probability of exceedance in 50 years (approximately a 975-year return period), it is used only to determine the parameters of the annual

maximum distribution. The mean and standard deviation adopted correspond to the annual maxima, ensuring consistency with the yearly reliability formulation. The coefficient of variation of the annual maximum traffic load can be obtained from through the following procedure.

The distribution of the maximum traffic load for a given reference period is modelled using a Gumbel distribution. According to the background document of Eurocode 1 [129], the following scale factor depending only on the reference period T_R is derived:

$$SF(T_R) = 1.05 + 0.116 \log_{10} T_R = 1.05 + 0.0504 \ln T_R \quad (75)$$

The characteristic load in EC1 [127] is defined as the load with a probability of exceedance equal to 0.05 in 50 years [129]. This corresponds to a return period of approximately 975 years. Therefore:

$$SF(975) = 1.397 \quad (76)$$

When modelling the annual maximum traffic load through its probability distribution, the probability of exceeding the characteristic load x_{kT_R} in one year can be expressed as:

$$P_{1kT_R} = 1 - \frac{1}{T_R} \quad (77)$$

Thus, the cumulative distribution function of the annual maxima satisfies:

$$F_1(x_{kT_R}) = P_{1kT_R} = 1 - \frac{1}{T_R} \quad (78)$$

On the other hand, using the scale factor formulation provided above, characteristic values associated with different reference periods are linked through:

$$\frac{x_{kT_R}}{SF(T_R)} = \frac{x_{k975}}{SF(975)} \quad (79)$$

Substituting the expression for $SF(T_R)$, and recalling that $T_R = \frac{1}{1 - F_1(x_{kT_R})}$, the previous relationship becomes:

$$\begin{aligned} \frac{x_{kT_R}}{SF(T_R)} &= \frac{x_{k975}}{SF(975)} \Rightarrow \\ x_{kT_R} &= \frac{x_{k975}}{SF(975)} (1.05 + 0.0504 \ln T_R) \\ &= \frac{x_{k975}}{SF(975)} \left(1.05 + 0.0504 \ln \left(\frac{1}{1 - F_1(x_{kT_R})} \right) \right) \end{aligned} \quad (80)$$

Rearranging, it is possible to obtain:

$$\frac{x_{k_{TR}} - x_{k_{975}} \left(\frac{1.05}{SF(975)} \right)}{x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right)} = -\ln(1 - F_1) \quad (81)$$

which leads to:

$$F_1(x_{k_{TR}}) = 1 - \exp \left(- \frac{x_{k_{TR}} - x_{k_{975}} \left(\frac{1.05}{SF(975)} \right)}{x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right)} \right) \quad (82)$$

This expression demonstrates that the annual maximum traffic load follows a Gumbel distribution with parameters:

$$\lambda_1 = x_{k_{975}} \left(\frac{1.05}{SF(975)} \right); \quad v_1 = x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right) \quad (83)$$

The mean, standard deviation, and coefficient of variation (where γ_E is the Euler–Mascheroni constant) are therefore:

$$\begin{aligned} \mu_1 &= \lambda_1 + v_1 \gamma_E = x_{k_{975}} \left(\frac{1.05}{SF(975)} \right) + x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right) \gamma_E \\ \sigma_1 &= \frac{\pi}{\sqrt{6}} v_1 = \frac{\pi}{\sqrt{6}} x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right) \\ V_1 &= \frac{\sigma_1}{\mu_1} = \frac{\frac{\pi}{\sqrt{6}} x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right)}{x_{k_{975}} \left(\frac{1.05}{SF(975)} \right) + x_{k_{975}} \left(\frac{0.0504}{SF(975)} \right) \gamma_E} \\ &= \frac{\frac{\pi}{\sqrt{6}} \left(\frac{0.0504}{SF(975)} \right)}{\left(\frac{1.05}{SF(975)} \right) + \left(\frac{0.0504}{SF(975)} \right) \gamma_E} \end{aligned} \quad (84)$$

It is important to note that the coefficient of variation V_1 is independent of the value of the characteristic load $x_{k_{975}}$. Substituting the numerical values:

$$\begin{aligned} V_1 &= \frac{\frac{\pi}{\sqrt{6}} \left(\frac{0.0504}{SF(975)} \right)}{\left(\frac{1.05}{SF(975)} \right) + \left(\frac{0.0504}{SF(975)} \right) \gamma_E} = \frac{\frac{\pi}{\sqrt{6}} \left(\frac{0.0504}{1.397} \right)}{\left(\frac{1.05}{1.397} \right) + \left(\frac{0.0504}{1.397} \right) 0.5772} \\ &= 0.0599 \sim 0.06 \end{aligned} \quad (85)$$

Therefore, the annual maximum traffic load derived from the EC1 background formulation results in a coefficient of variation of approximately 6%.

The same equations can be used to calculate the mean and standard deviation of the annual maximum traffic load once the value of $x_{k_{975}}$ is specified.

For example, for the tandem load associated with lane 1, the characteristic value is:

$$x_{k_{975}} = Q_{1k} = 150 \text{ kN} \quad (86)$$

The corresponding mean value μ_{Q_1} and standard deviation σ_{Q_1} of the annual maxima are:

$$\begin{aligned} \mu_{Q_1} &= 150 \left(\frac{1.05}{1.397} \right) + 150 \left(\frac{0.0504}{1.397} \right) 0.5772 = 115.9 \text{ kN} \\ \sigma_{Q_1} &= \frac{\pi}{\sqrt{6}} 150 \left(\frac{0.0504}{1.397} \right) = 6.94 \text{ kN} \end{aligned} \quad (87)$$

This procedure is consistently applied to define the probability distributions of all traffic load components used in the annual reliability assessment.

Uncertainties related to slab thickness (θ_s) are modelled through a uniform distribution ranging between 15 cm and 25 cm [96], while the external girder dimension uncertainties are disregarded as, based on [99], the standard deviation of the difference from the nominal value of external dimensions is about 1 cm. Moreover, based on [130], the beam spacing can be assumed deterministic when the number of girders composing the deck is known.

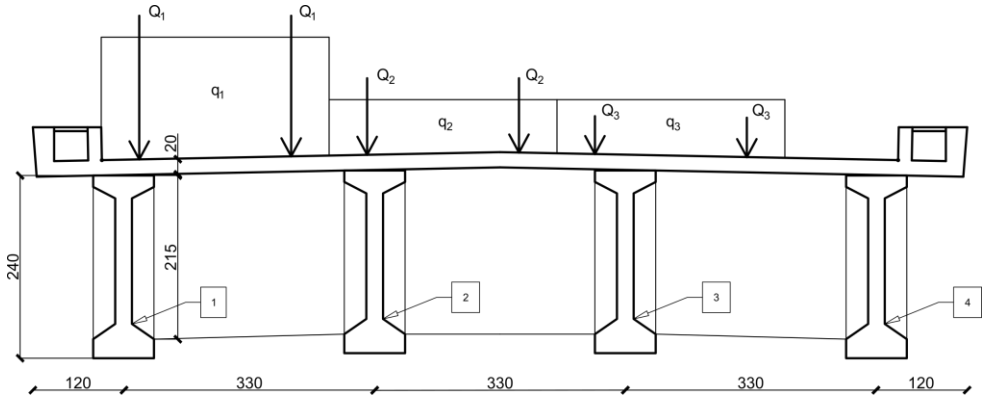


Figure 8-11: Load arrangement on the deck.

8.1.3 Corrosion Modelling

The deck is subjected to corrosion of both the post-tensioning cables and the shear reinforcements (stirrups). For both components, the activation and propagation phases are modelled, accounting for generalised and pitting corrosion, as introduced in Chapter 7.

In the case of tendons, the degradation process also includes the occurrence and location of defects, modelled as random variables following the approach in Section

5.4. As discussed in Section 7.3, two distinct conditions are considered: corrosion spots and corrosion spots in the presence of voids, which are associated with the variables C_{cr_s} and C_{cr_c} , respectively. Reliability analysis for tendon degradation is therefore performed as described in Chapter 6.

The carbonation model proposed by Silva et al. [115] is employed to avoid reliance on the carbon dioxide diffusion coefficient in concrete, which is difficult to estimate accurately [114], [131]. This model uses a statistical approach to describe carbonation, introducing two parameters, k_d and k_w , that depend on the external relative humidity. The time to generalised corrosion, T_g , is given by:

$$T_g(M_{ca}(\Theta_{ca})) := \begin{cases} T_g = \frac{c_c^2}{k_d^2} & RH \leq 70\% \\ T_g = \frac{c_c^2}{k_w^2} & RH > 70\% \end{cases} \quad (88)$$

where c_c is the concrete cover [mm] and RH is the relative humidity [%]. The parameters k_d and k_w are expressed as:

$$k_d = S_1 \cdot [CO_2] - S_2 \cdot EC - S_3 \cdot f_c + S_4 \quad (89)$$

$$k_w = S_5 \cdot [CO_2] - S_6 \cdot C_l - S_7 \cdot f_c + S_8 \quad (90)$$

In Eqs. (89) and (90), $[CO_2]$ is CO_2 content in the surrounding air [%], EC is the exposure class according to Eurocode: $EC := \{1 \text{ for } XC1; 2 \text{ for } XC3; 3 \text{ for } XC4\}$, C_l is the clinker content [kg/m^3], and f_c is the concrete compressive strength [MPa]. The eight regression parameters (S_n) are modelled as hyperparameters with values reported in Table 8-3, [115].

The prior distributions of both the dry ($p'_{k_d|RH \leq 70\%}$) and wet ($p'_{k_w|RH > 70\%}$) carbonation parameters are defined using nonparametric kernel density estimation, avoiding assumptions about the underlying distributions. The final prior distribution is obtained via the total probability theorem:

$$p'_k = p'_{k_d|RH \leq 70\%} \cdot F_{RH}(70\%) + p'_{k_w|RH > 70\%} \cdot (1 - F_{RH}(70\%)) \quad (91)$$

where F_{RH} the cumulative distribution function of the annual relative humidity.

Table 8-3. Carbonation model parameters

Parameter	Distribution	Mean	Standard Deviation
S1	N	0,556	0,024
S2	N	-3,602	0,482
S3	N	-0,148	0,033
S4	N	18,734	1,785
S5	N	3,355	0,088
S6	N	-0,019	0,003
S7	N	-0,042	0,013
S8	N	10,83	0,571

N: Normal.

The time to pitting corrosion, T_p , is instead evaluated using Eq. (38) defined in Chapter 7, and the propagation models follow those presented in Sections 7.2 and 7.3.1.

The set of corrosion variables includes: activation energy b_e , the ageing factor a_A , the air temperature T_{real} , the pitting factor α , the rate of an active corrosion $V_{corr,a}$ and the time of wetness ToW for the reinforcements, the corrosion current density i_c for the wires, the chloride surface concentration $C_{s,app}$, the threshold chloride concentration C_{cr_s} or C_{cr_c} , the chloride diffusion coefficient $D_{RCM,0}$, the relative humidity RH , and the carbon dioxide concentration $[CO_2]$. For tendons specifically, the variables $\rho_{CV}|\mathbb{D}$ and $\zeta_{CV}|\mathbb{D}$, are also considered.

8.1.3.1 Correlation modelling

Spatially correlated variables are modelled using RF [29], with the models from [30] used to define both the correlation function and the correlation length. A Gaussian correlation function with a correlation length of 2 m is adopted for all variables, consistent with the robustness demonstrated in [30]. Variables modelled with spatial dependency include the chloride surface concentration $C_{s,app}$, the critical chloride threshold C_{cr} , the chloride diffusion coefficient $D_{RCM,0}$, and the concrete compressive strength f_c . These RFs are summarised in Table 8-4.

Moreover, the yield strengths of stirrups and cables are assumed to have a correlation coefficient of 1 within the same girder and 0.4 across different girders [99]. Environmental variables (e.g., T_{real} , RH , etc), traffic loads, and model uncertainties are modelled as fully correlated. Perfect correlation among girders is modelled for the load variables, since the deck is subjected to the same traffic load realisation. Model uncertainties are also treated as fully correlated, because identical mechanical models are employed to estimate both capacity and demand for all girders. Finally, the girders are subjected to the same environmental conditions, and these variables are therefore modelled as fully correlated. Whether spatially distributed environmental measurements would be available, the latter hypothesis

could be relaxed by introducing partial correlation between girders. All other variables are modelled as independent.

The remaining probabilistic models used in the analyses are presented in Table 8-5.

Table 8-4: Variables modelled as RF to account for spatial correlation

Variable	RF distribution	Mean	Standard Deviation	Correlation type	Correlation length [m]
C_{cr_c} [30]	B [0.2 ; 2]	0,6	0,15	Gaussian	2
C_{cr_s} [30]	LN [0.2 ; 2]	1,25	0,37	Gaussian	2
$C_{s,app}$ * [30]	NP	μ_{Cs}	0,68 or 1,13	Gaussian	2
$D_{RCM,0}$ [30]	N	498.27	99.65	Gaussian	2
f_c [30]	LN	53.5	6.1	Gaussian	2

B: Beta; LN: Log-Normal; NP: Non-Parametric. *The mean value of the apparent chloride concentration is modelled as an hyperparameter, [67]. Its standard deviation is equal to 0,68 or 1,13, respectively for the stirrups or the tendons.

Table 8-5: Probabilistic models used to perform the System Reliability Analysis

Variable	Distribution	Mean	Standard Deviation	Reference
b_e [K]	N	4800	700	[67]
T_{ref} [K]	D	293	-	[67]
T_{real} [K]	N	288	8	[67]
a_A (Ageing factor)	B	0.3	0.12	[67]
t_0 [years]	D	0.0767	-	-
d_0 [mm]	D	7	-	-
ϕ_w [mm]	D	12	-	-
c_w [mm]	D	30	-	-
u_w [mm]	D	150	-	-
μ_{Cs} [wt.-%/b]	U [0.5 ; 1.5]	1.0	0.29	[67]
$C_{s,app}$ [wt.-%/b]	NP	μ_{Cs}	$\mu_{Cs} * 0.45$ (stirrups) $\mu_{Cs} * 0.75$ (cables)	[67]
C_{cr_c} [wt.-%/c]*	B [0.2; 2]	0,6	0,15	[67]
C_{cr_s} [wt.-%/c]*	LN [0.2; 2]	1,25	0,37	[132]
$D_{RCM,0}$ [mm ² /year]	N	$8.9 * 10^{-12}$	$1.78 * 10^{-12}$	[67]
K_c	N	1	0.1	[52]
α (pitting factor)	LN	9.28	4.04	[67]
i_c [$\mu A/cm^2$]	N	1	0.2	[133]
$V_{corr,a}$ [mm/year]	W	0.03	0.02	[65]
ToW	N	0.5	0.15	[65]
c_c (cover)[mm] **	U [25 ; 35] or [15 ; 27]	30 or 21	2.88 or 3.46	[56]
$\rho_{CV} \mathbb{D}$	Copula	Sec. 5.4.2	Sec. 5.4.2	Sec. 5.4.2
$\zeta_{CV} \mathbb{D}$	Copula	Sec. 5.4.3	Sec. 5.4.3	Sec. 5.4.3

Application of the Framework to Case Studies

k_t	D	1	-	[67]
EC	D	2	-	-
RH [%]	W [0;100]	75	15	[113]
$[CO_2]$ [%]	D	0.04	-	[134]
C_l [kg/m ³]	D	380	-	-
f_y [MPa]	N	1414	35.35	[99]
f_{yw} [MPa]	N	510	51	[99]
f_c [MPa] ***	LN	53.5	6.1	-
P_{tot} [kN]	N	1370	137	[99]
θ_d [cm]	N	1	1	[99]
θ_s [cm]	U	20	3	[96]
$\theta_{M,C}$ (Capacity model unc.)	LN	1.2	0.15	[99]
$\theta_{M,D}$ (Demand model unc.)	LN	1	0.1	[99]
$\theta_{V,C}$ (Capacity model unc.)	LN	1	0.1	[99]
$\theta_{V,D}$ (Demand model unc.)	LN	1	0.1	[99]
q_g [kNm] ****	N	55	7.15	-
Q_1 [kN] *****	G	115.9	6.94	[127]
q_1 [kN/m ²] *****	G	6.95	0.42	[127]
Q_2 [kN] *****	G	77.2	4.63	[127]
q_2 [kN/m ²] *****	G	1.93	0.12	[127]
Q_3 [kN] *****	G	38.6	2.31	[127]
q_3 [kN/m ²] *****	G	1.93	0.12	[127]

N: Normal; D: Deterministic; LN: Log-normal; W: Weibull; B: Beta; U: Uniform; NP: Non parametric; G: Gumbel. * Depending on whether the analysis is performed in good or poor concrete conditions. ** Considered depending on duct size for no void cases, and according to [56], for corrosion in the presence of voids. *** Evaluated considering 44 MPa as the 5% characteristic value, provided by the original design documents, a coefficient of variation equal to 0.114 is adopted based on [130]. **** Evaluated considering a coefficient of variation equal to 0.05. ***** Evaluated considering a coefficient of variation of the Eurocode 1 load scheme 1 equal to 0.06 based on [129] and Eq. (85).

8.1.4 Results and discussion

The reliability of the crossbeams is first assessed and compared with the reliability of the main girders. The crossbeam time-dependent reliability index ($\beta_{Crossbeam}$) is shown together with the reliability index of the external main girder ($\beta_{Girder1}$) in Figure 8-12.

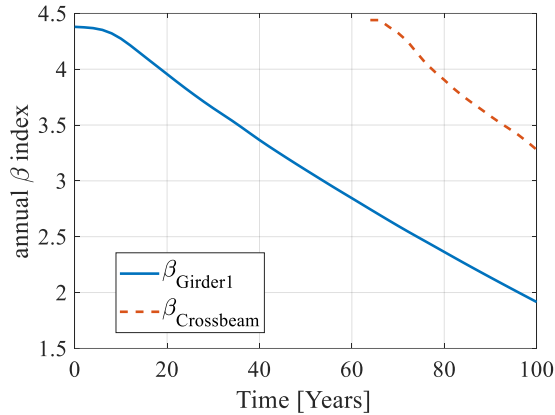


Figure 8-12: Time dependent reliability index of the crossbeams and of Girder 1

Figure 8-12 shows that the reliability index of the crossbeams is consistently higher than that of the main girder throughout the entire analysis period. Consequently, the probability of failure of a crossbeam remains more than an order of magnitude lower than that of a girder at any time.

Similarly, the series system reliability is evaluated. In particular, the system reliability is assessed under two configurations: (i) considering a series system composed of both girders and crossbeams; and (ii) considering a series system composed only of the girders. Correlation among components is modelled using random fields for the degradation-related variables and for the concrete compressive strength, while model uncertainty variables and traffic loads are assumed to be fully correlated, as discussed in Section 8.1.3. The results are shown in Figure 8-13.

Figure 8-13 compares the annual system reliability index evaluated without accounting for crossbeam failure and that obtained when crossbeam failure is included as an additional system component. The figure shows that the estimated annual reliability is virtually unchanged throughout the entire analysis period.

Therefore, the crossbeams can be considered to retain adequate reliability for transferring loads between the girders over the analysed service life.

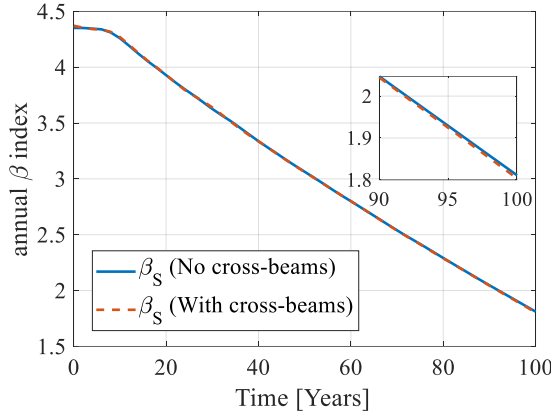


Figure 8-13: System reliability with or without accounting for crossbeam failure

This result supports the modelling assumption adopted within the brittle Daniels System mechanical model. Since the crossbeams exhibit reliability indices significantly higher than those of the longitudinal load-carrying members, the governing system failure is controlled by the longitudinal girder behaviour. Accordingly, neglecting crossbeam failure (i.e., treating crossbeams as non-governing components in the system formulation) does not introduce a bias in the probabilistic assessment. Together with the NLFE comparisons (Section 8.1.1.3), this indicates that the combined Daniels/Courbon approximation is computationally efficient and only moderately conservative (~ -8 to -18%) for the studied PT deck.

This verification step is essential within a time-variant reliability framework, as it demonstrates that the simplifying assumption underlying the analytical system model is mechanically consistent over the entire timespan considered for this case study.

Following this verification, the spatial and time-dependent system reliability analysis is performed according to the proposed framework. In particular, two different tendon degradation models are employed. In the first case, the probabilistic model developed in Chapter 5 is applied, while in the second case, defects are assumed to be continuously present along the entire deck length, as commonly assumed in the literature (see, for instance, [56], [57]). This comparison highlights the advantages of using the developed probabilistic model, which treats defect occurrence and location as random variables. Figure 8-14a compares the annual reliability indices for the series system model calculated with the developed model, β_S , and with the deterministic assumption, $\beta_{S,w}$. Figure 8-14b shows a similar comparison for the brittle DS model (Eq. (3)) calculated in both cases (β_{bDS} or $\beta_{bDS,w}$). Moreover, the figure also shows the target reliability index for CC2 ($\beta_{t,CC2}$)

and CC3 ($\beta_{t,CC3}$), used to estimate lifetime in the case of major repair (CC2) and collapse (CC3), respectively (see Section 8.1).

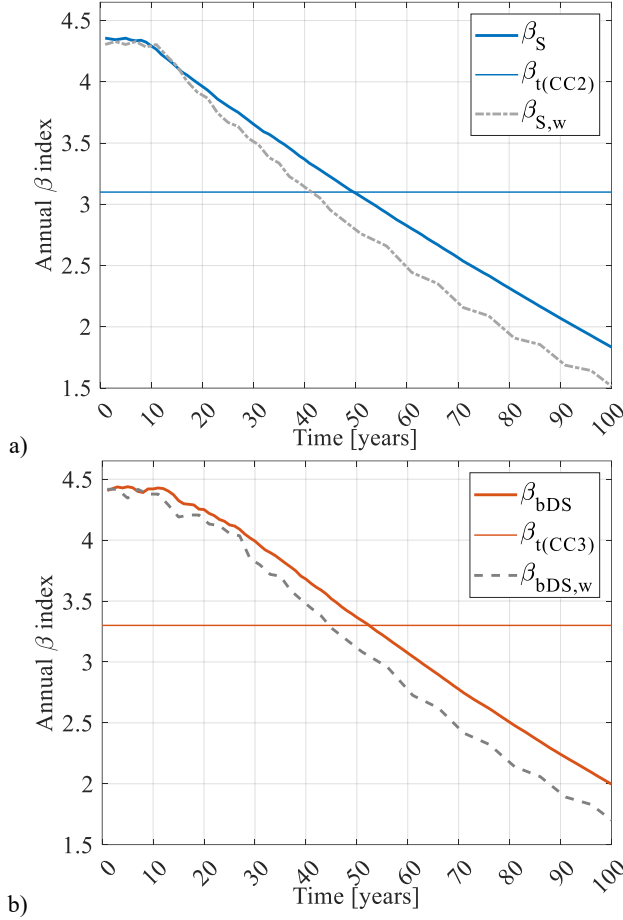


Figure 8-14: Comparison of annual reliability indices with probabilistic tendon degradation modelling versus deterministic assumption: a) series system (β_S or $\beta_{S,w}$); b) brittle Daniels System (β_{bDS} or $\beta_{bDS,w}$).

The technical service life before major repair (Figure 8-14a) is estimated at 50 years using the developed probabilistic model and 41 years under the deterministic assumption, corresponding to a difference of nine years (approximately 20%). For global collapse (Figure 8-14b), the corresponding values are 53 and 45 years, respectively, resulting in an extension of eight years (approximately 15%). The difference becomes even more pronounced when considering only the bending failure mode, the failure mode most strongly influenced by tendon degradation. For instance, Figure 8-15 presents the comparison of the annual reliability index for the series system subjected to bending failure only, evaluated either assuming

deterministically placed defects, $\beta_{S_M, w}$, or using the developed probabilistic model, β_{S_M} .

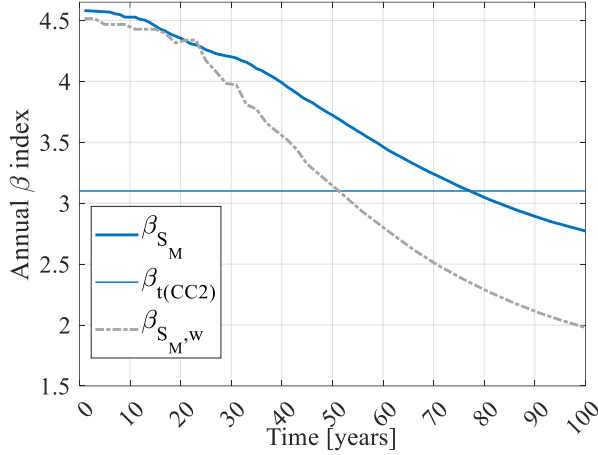


Figure 8-15: Comparison of annual reliability indices with probabilistic tendon degradation modelling (β_{S_M}) versus deterministic assumption ($\beta_{S_M, w}$) for the bending failure mode of the series system.

Based on Figure 8-15 the threshold $\beta = 3.1$ is reached 26 years earlier when defects are assumed to be deterministically located. This corresponds to a reduction in the estimated service life of approximately -30% compared to the developed model.

These results confirm that the developed model can extend service-life estimates, providing a more rational basis for maintenance planning and resource allocation, and reducing infrastructure maintenance costs.

Despite accounting for the system behaviour of the structure, the 53-year estimate for collapse is close to the 50-year design life typically expected for Italian bridges built during the economic boom. Furthermore, the estimate can be extended by including survival information, given that the bridge is already 49 years old (see Section 4.3).

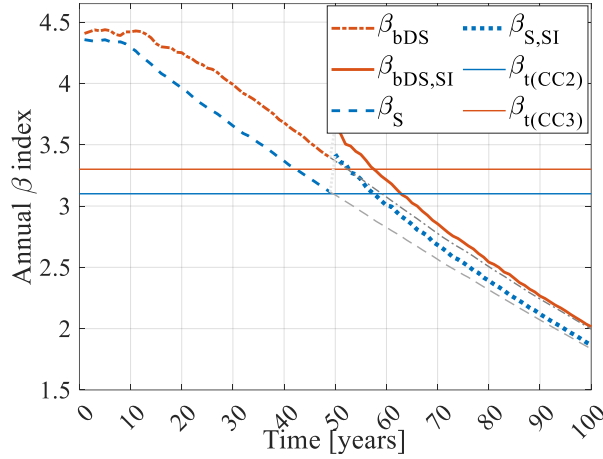


Figure 8-16: Reliability indices updated using the Survival Information in both brittle DS ($\beta_{bDS,SI}$) and series system ($\beta_{S,SI}$) configurations.

Figure 8-16 illustrates the updated reliability indices obtained by conditioning on survival. The effect is significant: service life increases by 9 years for major repair and 5 years for collapse. This means the bridge can continue to operate well beyond its nominal design life, with direct implications for cost savings and sustainability, as interventions can be postponed without compromising safety.

8.1.4.1 Sensitivity analysis

To identify the most influential variables, a sensitivity analysis is performed by calculating Spearman correlation coefficients (ρ) for each input variable relative to the LSF.

Figure 8-17 presents the results for the most loaded girder, considering both shear and bending failure modes. Only the twelve most sensitive variables are shown for clarity.

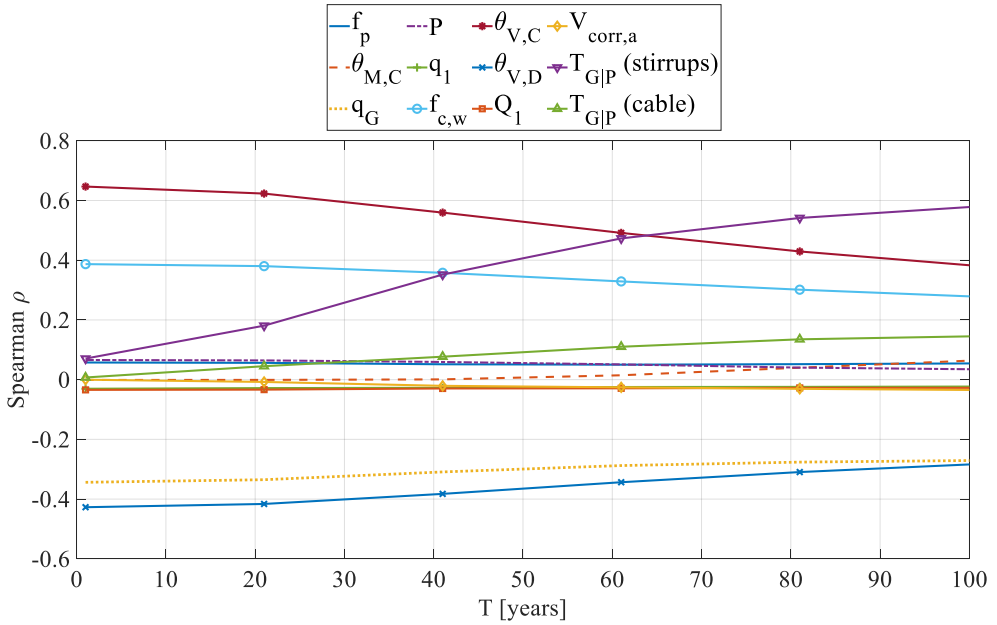


Figure 8-17: Sensitivities of the LSF considering both shear and bending failure modes for the most loaded girder.

Figure 8-17 reveals that the shear capacity model uncertainty ($\theta_{V,C}$) and the concrete compressive strength ($f_{c,w}$) dominate on the capacity side, while the shear demand model uncertainty ($\theta_{V,D}$) and the permanent load (q_G) dominates on the demand side. Prestressing force (P) has a small positive effect, counteracting shear load, but its influence decreases over time due to tendon degradation. As expected, corrosion-related variables ($V_{corr,a}$, $T_{G|P}(stirrups)$, $T_{G|P}(cable)$) gain importance as the analysis progresses. In particular, stirrup initiation time ($T_{G|P}(stirrups)$) becomes the dominant variable after year 60 and shows a significant influence ($\rho > 0.2$) as early as year 23, underlining its critical role even in the early stages of degradation.

Moreover, the figure indicates that shear failure dominates in the early years, as reflected by the higher absolute values of shear-related sensitivities. Over time, however, bending failure becomes increasingly important (as also recognised in [96]), as evidenced by the increasing influence of bending-related variables ($\theta_{M,C}$ and $T_{G|P}(cable)$) in later stages.

To isolate the factors most relevant to bending failure, a second sensitivity analysis is conducted considering only the bending LSF.

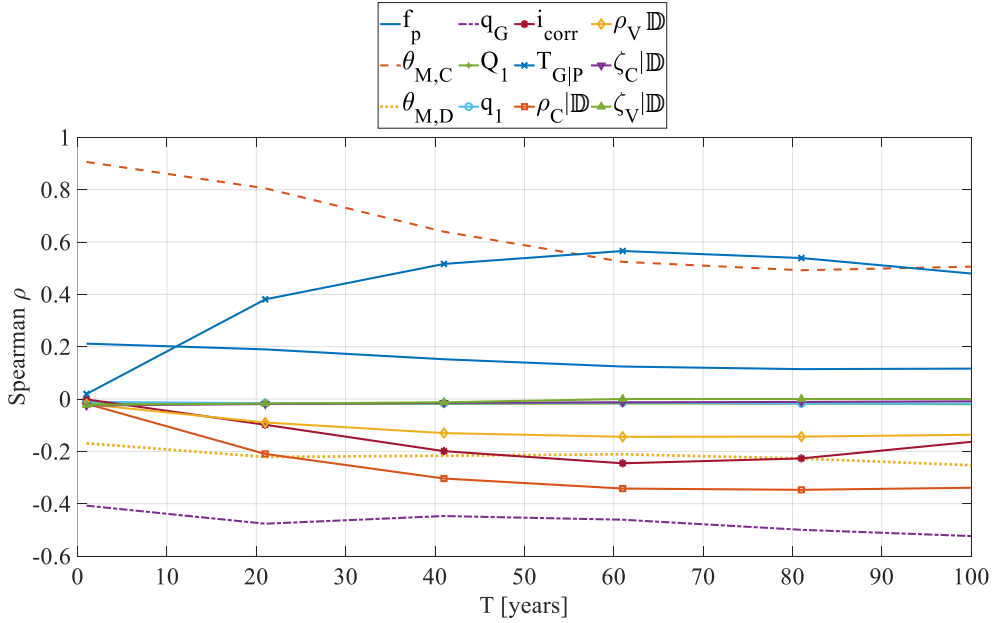


Figure 8-18: Sensitivities of the LSF considering only bending failure mode for the most loaded girder.

Figure 8-18 shows that capacity ($\theta_{M,C}$), the permanent load (q_G), and demand model uncertainties ($\theta_{M,D}$) dominate in the early years. On the capacity side, the rupture stress of wires (f_p) is also highly influential. On the demand side defect location variables ($\zeta_C|D$ and $\zeta_V|D$) are initially important but lose significance as degradation progresses, while defect occurrence variables ($\rho_C|D$ and $\rho_V|D$) become dominant in later years. For example, $\rho_C|D$ becomes the second most sensitive, on the negative side, variable after year 21, reflecting the increasing importance of the number of corrosion spots and voids over their precise location. Corrosion propagation and initiation variables (i_{corr} , $T_{G|P}$) also grow in significance, with $T_{G|P}$ becoming the dominant factor between years 58 and 94, and the second-most influential variable as early as year 41.

8.1.4.2 Spatial distribution of failure sections

The influence of the spatial variability of corrosion on failure locations is illustrated in Figure 8-19. The violin plots show the distribution of failed sections every 20 years, for both bending (Figure 8-19a) and shear failure (Figure 8-19b). For bending, failures occur predominantly near midspan (≈ 21 m) before degradation (up to year 30). At year 40, early signs of degradation lead to a broader distribution, and by year 60, failures also appear near supports, a scenario typically disregarded in engineering assessments due to low bending effects in those zones.

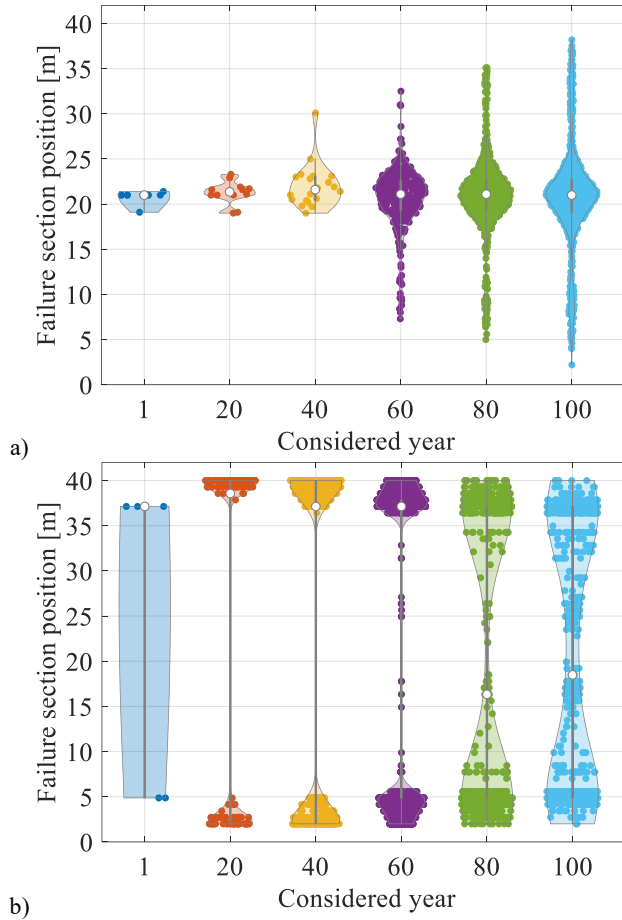


Figure 8-19: Position of the failed sections for a) bending failure mode; b) shear failure mode.

For shear, failures are concentrated within 6 m of the supports, with clustering around 5 m, where shear stress is high but the contribution of the concrete strut (see Eq. (73)) is significantly reduced. From year 30 onward, failures spread farther along the girder, and shear failure is also observed at sections where the stirrup area is reduced (see Figure 8-1d), indicating that highly degraded girders may also fail near midspan and in regions where the stirrup spacing increases or the stirrup diameter is reduced. These results underscore a key management implication: failure is not limited to peak load locations but is increasingly driven by scattered degradation after year 30. For well-preserved bridges, inspections can focus on peak load zones. As degradation progresses, however, inspection campaigns must gradually expand to cover larger portions of the structure, and for highly degraded bridges, they should include the entire element.

Based on the results provided in Figure 8-19a and b, inspection zones are identified at different degradation stages for simply supported girders similar to the one studied. The identified zones cover either 90% (darker shaded zones in Figure 8-20 and Figure 8-21) or 95% (lighter shaded zones in Figure 8-20 and Figure 8-21) of the probability of failure-section occurrence. Figure 8-20 illustrates the inspection zones in the case of bending failure. For the undamaged case, the midspan region covers virtually 100% of the possible failure sections. For limited corrosion (years 20 and 40 in the analysis), inspecting about 15% of the girder length centred at midspan allows 95% of the possible failure sections to be covered. As degradation becomes moderate (year 60 in the analysis), it is necessary to expand the inspection zone to about 20% and 36% of the girder length around midspan in order to cover 90% and 95% of the failure sections, respectively. Finally, in the case of advanced corrosion (years 80 and 100 in the analysis), it becomes necessary to inspect additional zones located approximately at the quarter points of the girder in order to achieve the desired coverage, leading to a total inspection length of about 55% and 65% of the girder length for the two considered coverage levels.

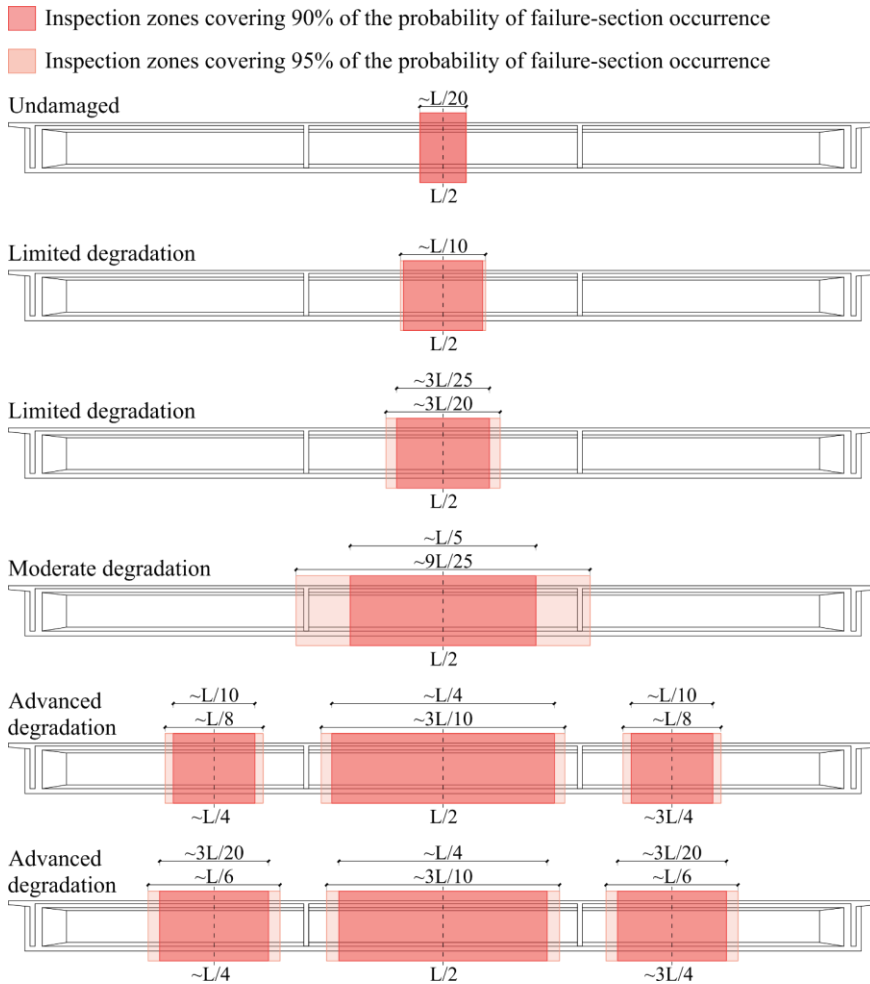


Figure 8-20: Girder zones to inspect at varying degradation stages for bending failure

Similarly, Figure 8-21 shows the inspection zones in the case of shear failure. For the undamaged case, sections located at approximately 12% of the girder length from each support should be inspected to cover virtually 100% of the failure sections. This corresponds to the region where the contribution of the concrete strut in the shear-capacity model is reduced, while the shear demand remains high. For limited corrosion (years 20, 40, and 60 in the analysis), inspecting two symmetric zones, each covering about 10% of the girder length, centred at a distance of about 10% of the span from each support, allows 95% of the possible failure sections to be covered. As degradation becomes moderate (year 80 in the analysis), the inspection zones must be expanded to about 20% and 25% of the girder length, centred approximately at 15% of the span from both supports, in order to cover 90%

and 95% of the failure sections, respectively. Finally, in the case of advanced corrosion (year 100 in the analysis), it becomes necessary to inspect additional central regions, located approximately at 40% and 60% of the span, leading to a total inspection length of about 50% and 70% of the girder length for the two considered coverage levels.

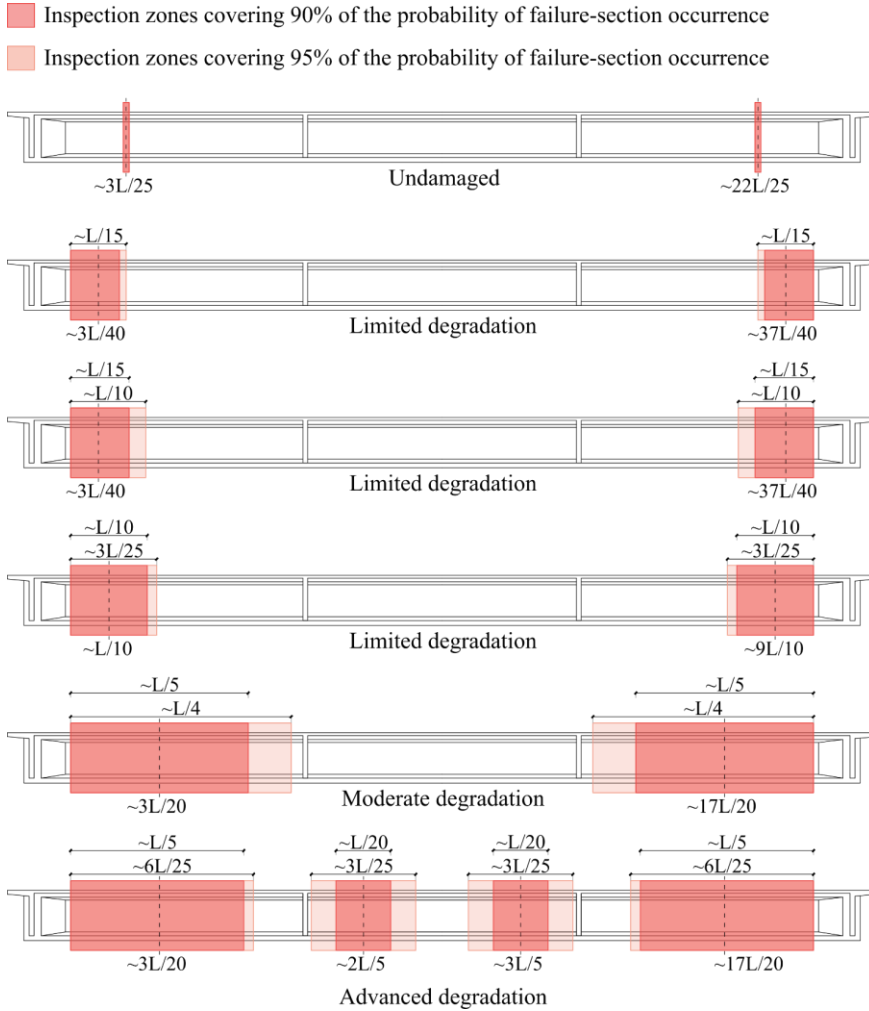


Figure 8-21: Girder zones to inspect at varying degradation stages for shear failure

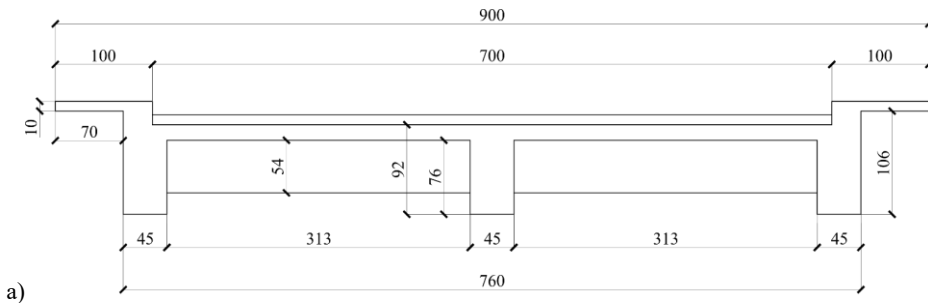
8.2 Reinforced concrete bridge

The system reliability of a 60-year-old RC bridge crossing the Arno River in the Province of Pisa is assessed. The bridge underwent testing campaigns and traffic-load monitoring between 2021 and 2023, and the resulting data are used to update the input variables and models. The bridge spans 130 m in total and consists of three RC arch segments (see Figure 8-22). These arches support half-joint decks composed of three simply supported RC girders, each with a span of 11.7 m. The deck has a total width of 9 m, including 7 m for carriageways and 1 m-wide walkways on each side. Figure 8-24 illustrates a schematic representation of the half-joint deck.



Figure 8-22: Picture of the studied bridge

The half-joint deck represents a critical structural element due to its lack of redundancy; hence, the reliability analysis focuses on this element to estimate the bridge's technical service life. The central and outer beams have heights of 92 cm and 116 cm, respectively. The central beam is reinforced with 10 ϕ 30 longitudinal bars arranged in three layers, while the outer beams contain 8 ϕ 30 longitudinal bars arranged in two layers. Shear reinforcement is provided at 20 cm spacing along the beam length, except at the half joints, where the spacing is reduced to 10 cm, and by inclined reinforcements (see Figure 8-23).



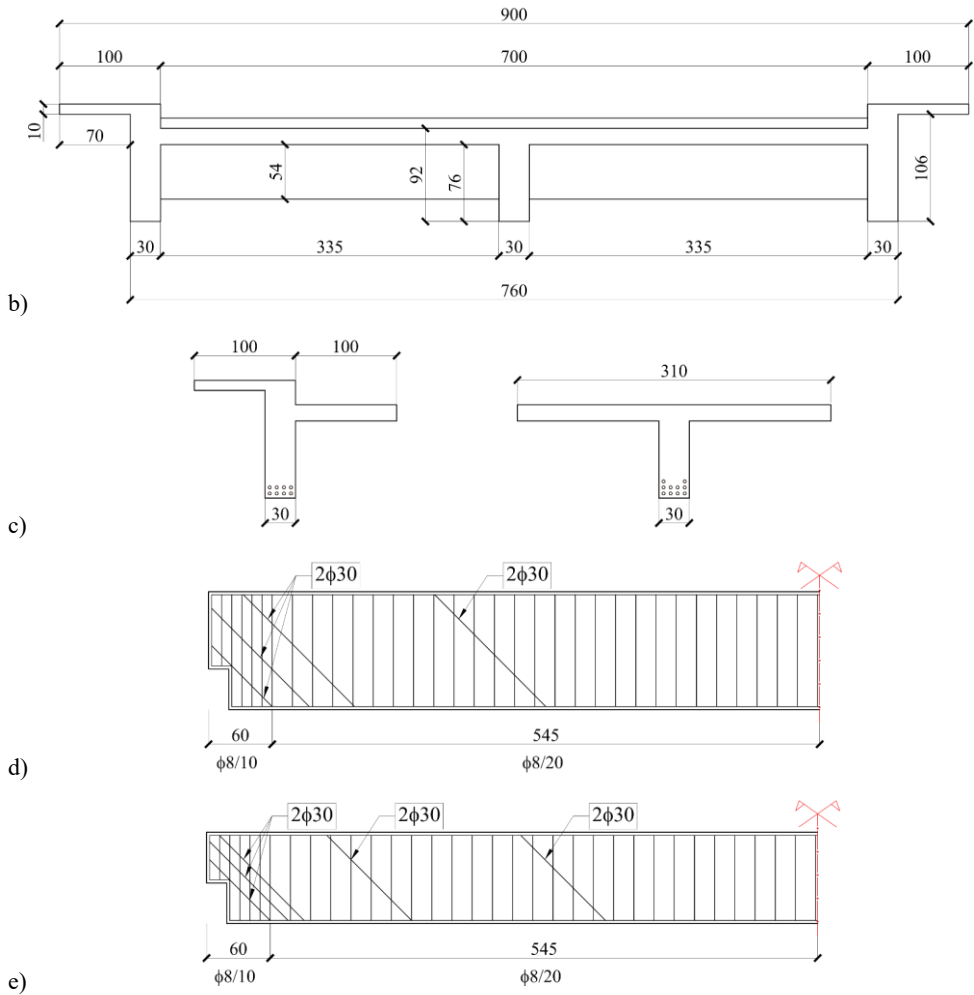


Figure 8-23: a) Cross-section near the half-joint section; b) midspan cross-section of the deck; c) midspan cross-section and reinforcement detail of the external (left) and internal (right) girders; d) external girder shear reinforcement layout; e) internal girder shear reinforcement layout.

The test campaigns and monitoring data acquired between 2021 and 2023 include compressive concrete strength, carbonation parameters, chloride content, and traffic loads. These measurements are directly incorporated into the reliability analysis to reduce uncertainty in the input models.

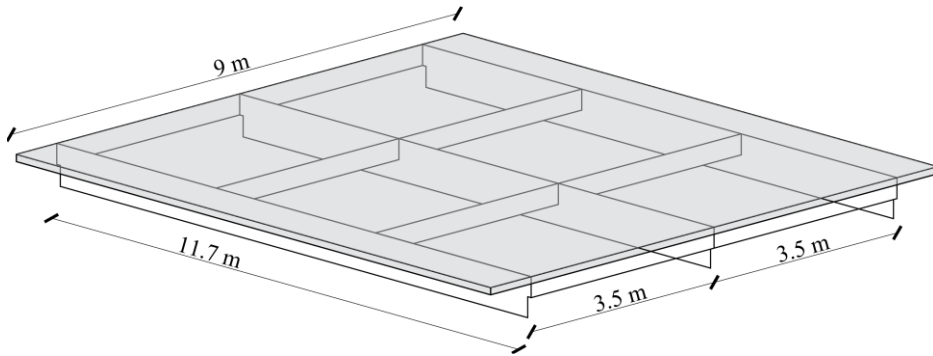


Figure 8-24: Half-joint deck.

Three main failure modes are considered: bending, shear, and half-joint failure. The deck is represented as a nested system (see Figure 8-25) that models failure at multiple levels: sections, girders, and the entire deck. Correlations among variables are modelled using a correlation structure drawn from relevant literature (e.g., [99]), and through RFs [29] to account for spatial variability.

Again, the configuration of the deck system depends on the consequence scenario. For partial collapse (major repair), the deck is modelled as a series system (Figure 8-25b). For global collapse, it is represented as a series of multiple brittle Daniels Systems [19], [22], (Figure 8-25c). These scenarios correspond to different failure consequences and therefore to different target reliability levels. According to the Italian guidelines, the target reliability index is defined on an annual basis. A target annual reliability index equal to 2.8 is specified, and when the calculated annual reliability index falls below this threshold, major repair interventions are required. This threshold is adopted as the target reliability index for Consequence Class 2 ($\beta_{t,CC2}$). Global collapse corresponds to Consequence Class 3, with an associated target reliability index of 3.3 ($\beta_{t,CC3}$), reflecting the high consequences of failure as discussed in Chapter 2.

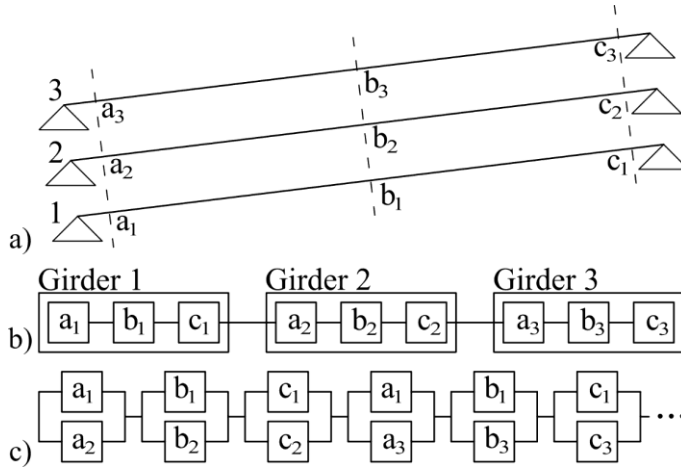


Figure 8-25: a) Schematic representation of a bridge deck, b) series system configuration, and c) series of brittle Daniels Systems.

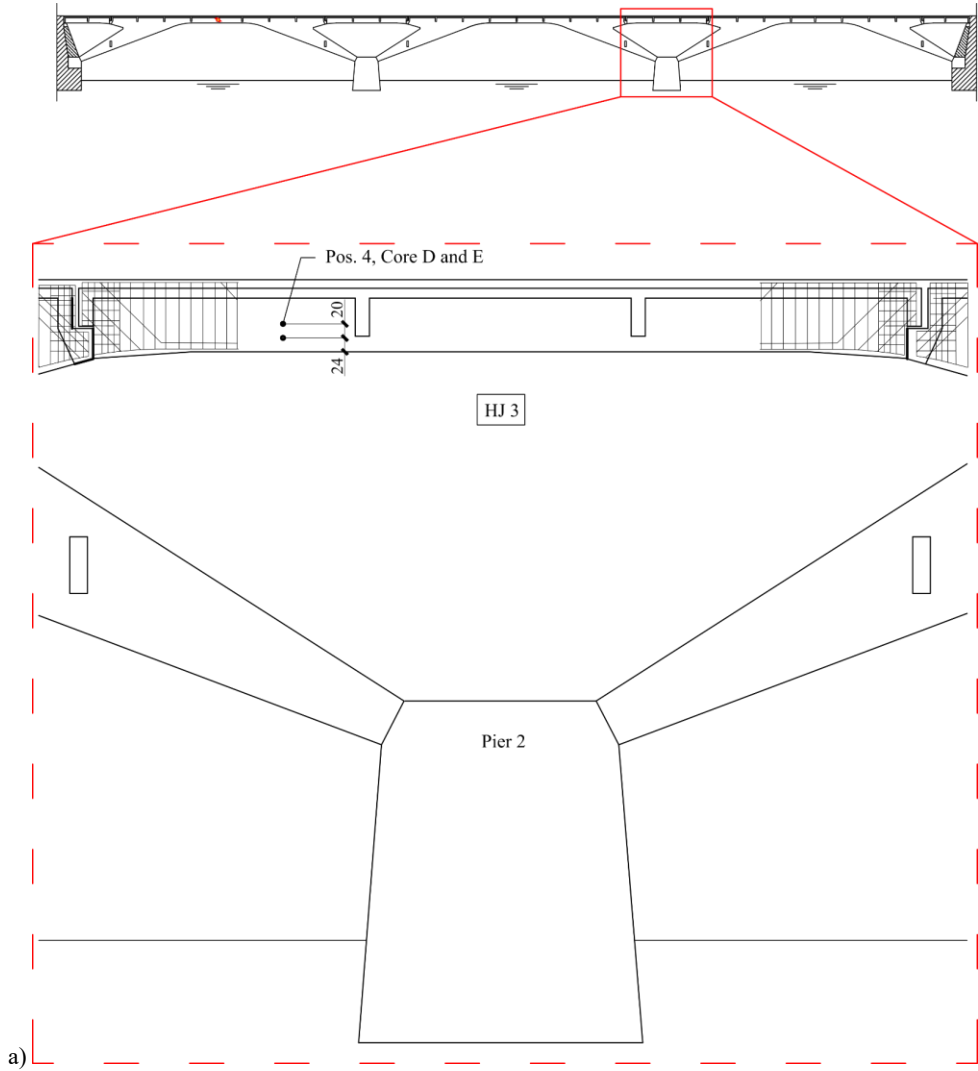
Similarly to the procedure described in Section 8.1.1, the approximation introduced by employing the DS modelling for the evaluation of global capacity is quantified through comparison with a NLFE model of the deck under two load configurations and four corrosion levels.

8.2.1 Test and monitoring campaign

A thorough test campaign was designed and performed to gather information about the bridge and to update the probabilistic models of some of the input variables. The test campaign was also accompanied by traffic monitoring, aimed at improving the characterisation of the traffic load variable. In particular, the campaign comprised: (i) the extraction of seventeen concrete cores, used to estimate the concrete compressive strength and carbonation depth; (ii) the extraction of fifty-six micro-cores for the estimation of the chloride content within the concrete; and (iii) the acquisition of traffic data through a Weight in Motion (WIM) system installed on the road leading to the bridge. The tests were distributed along the entire deck, and excerpts from the original test campaign drawings are illustrated in Figure 8-26.

Moreover, non-destructive tests (SonReb, pull-out, and Windsor tests) were carried out across the entire deck to evaluate whether the concrete quality is consistent throughout the structure. The results showed that the concrete properties do not vary significantly between different elements or regions of the structure.

The WIM system was installed near, but not directly on, the bridge. Indeed, due to the sensitivity of the WIM systems to pavement conditions, the installation location was carefully selected, avoiding crossroads and rough surfaces to ensure data reliability.



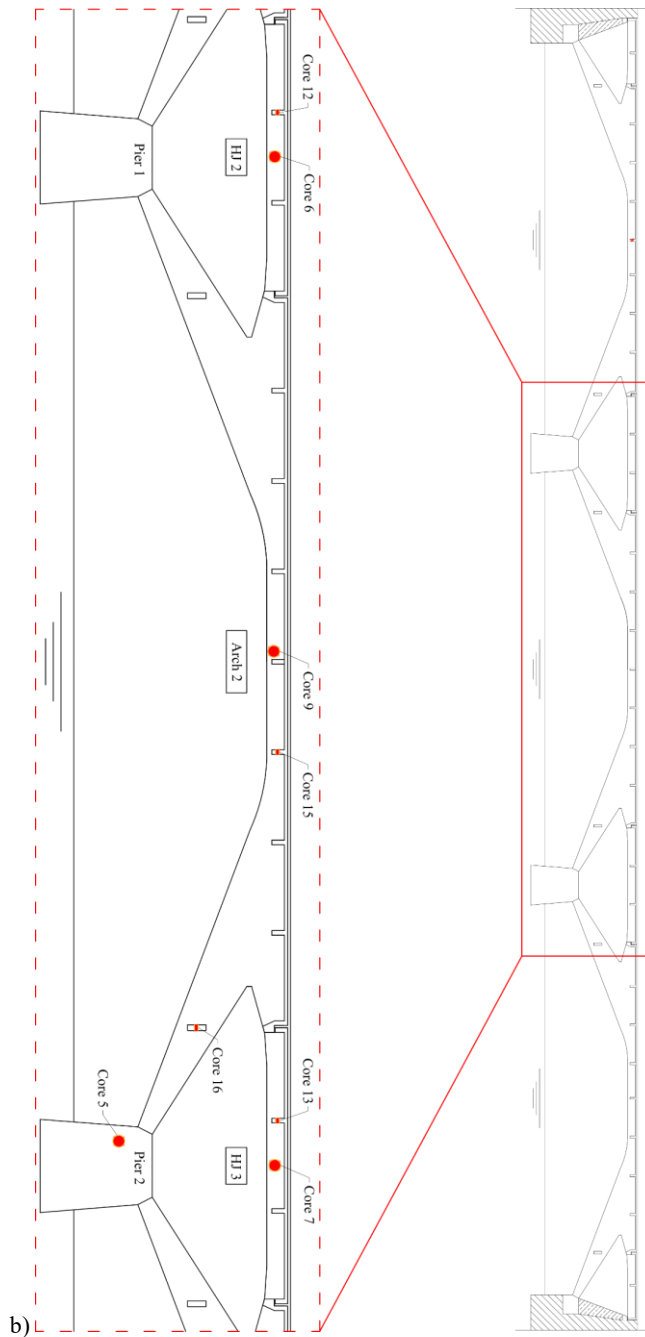


Figure 8-26: Excerpt from the original drawings utilised for the test campaign: a) micro-cores D and E utilised for the estimation of the chlorides concentration within the concrete; b) concrete cores utilised for concrete compressive strength and carbonation depth estimation.

8.2.2 Nonlinear Finite Element model of the RC deck

A NLFE model with lumped plasticity of the considered bridge deck is developed using the SAP2000 software. Girders and crossbeams are modelled using beam elements, while the slab is modelled using shell elements (Figure 8-27). Again, the slab is included for load introduction, while its stiffness is reduced to avoid double counting its contribution to the structural capacity.

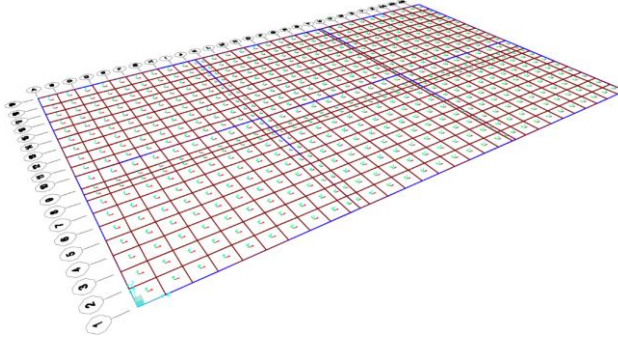


Figure 8-27: Developed grillage model

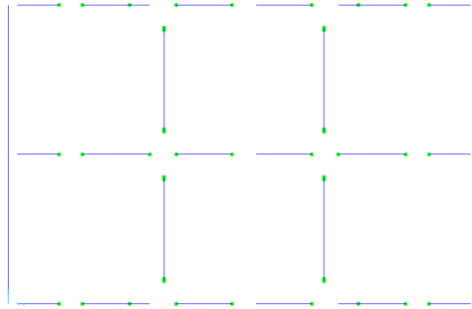


Figure 8-28: Positions of the modelled plastic hinges

Nonlinearity is modelled through plastic hinges, which are placed at the ends of the beam elements and at locations where changes in the reinforcement layout occur (see Figure 8-28 and Figure 8-29). As discussed in Section 8.1.1, the mean values of both the concrete compressive strength and the steel yield strength are employed to define the plastic hinges. Moreover, these are defined for four corrosion cases: no corrosion, and reinforcement area reductions of 10%, 20%, and 30%. To account for the reduced ductility of the plastic hinge in the presence of corrosion, the mechanical properties of the reinforcement are modelled as a function of the corrosion level following the model introduced in Section 7.2.

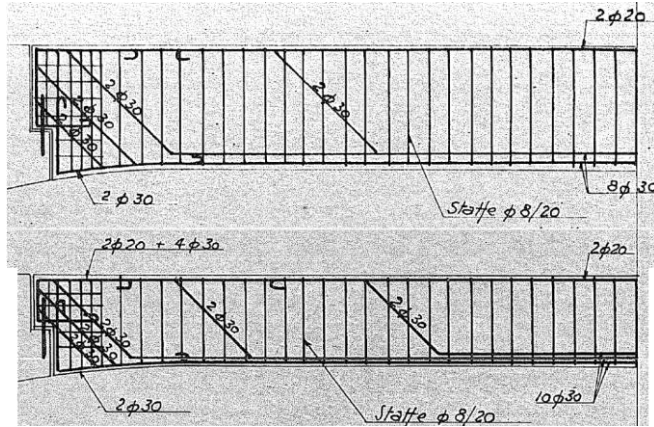


Figure 8-29: Excerpt from the original documentation of the reinforcement layout. Top: external girder. Bottom: inner girder.

The plastic rotation is estimated modelling the hinge length equal to the depth of the beam section [91], and Figure 8-30 shows the Moment-Curvature diagrams for the values of $\xi_s = [0, 10\%, 20\%, 30\%]$ for the midspan section of the external girder.

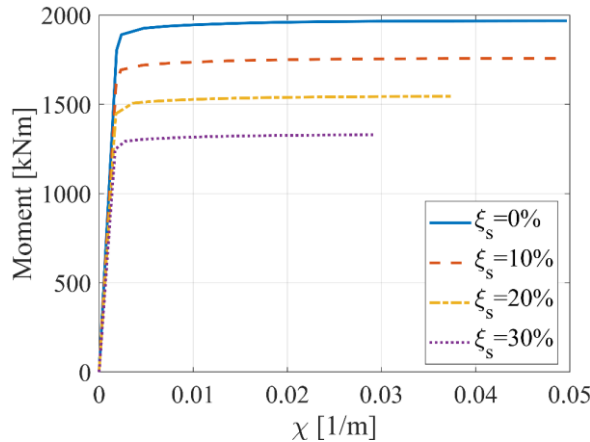


Figure 8-30: Moment-Curvature diagrams for the midspan section of the external girder at different corrosion levels

8.2.2.1 Load configuration

In order to investigate the effect of the load position on the global capacity, two cases are considered (see Figure 8-31): (i) the tandem load is placed at the midspan section, and the distributed load is applied over the entire carriageway; (ii) the tandem load is placed 1.2 metres from the support, i.e. at the end of the D-zone, and the distributed load is applied to the carriageway beyond the initial 1.2-metre region.

Again, the geometry and magnitudes of the load scheme corresponds to Load Model 1 of Eurocode 1 [127].

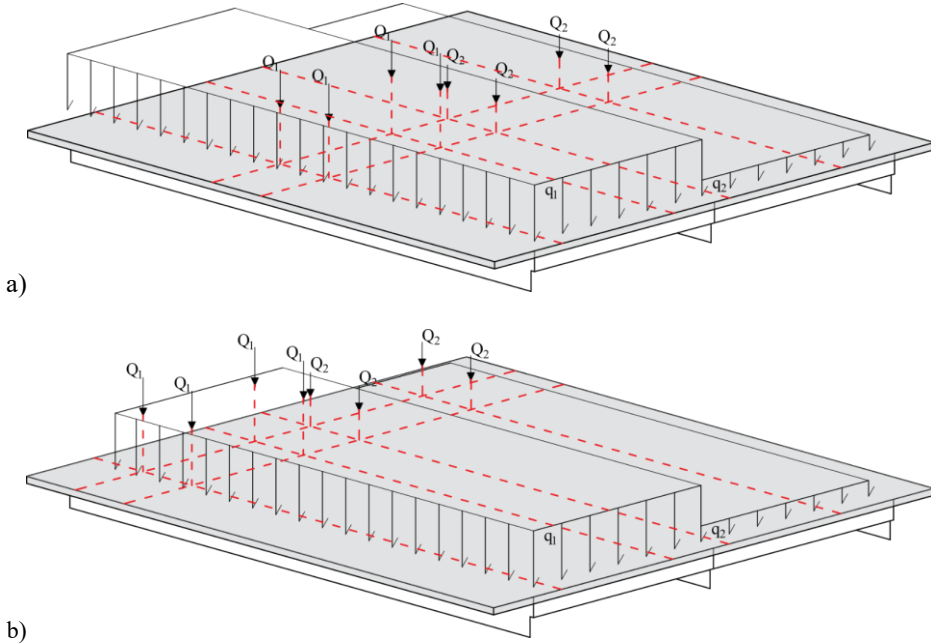


Figure 8-31: Schematic representation of the considered load configurations on the deck: a) the tandem load located at midspan; b) the tandem load located 1.2 metres from the support.

8.2.2.2 Outcomes of the NLFE analysis

A nonlinear static (push-down) analysis is performed in force control. The load intensity is gradually increased until convergence can no longer be achieved, i.e. until the stiffness matrix becomes singular.

A total of eight push-down analyses are carried out, four for each load configuration, by varying the corrosion degree of one of the external girders. In particular, the hinges applied to the most heavily loaded external girder (Figure 8-31) are modified according to the moment–curvature diagrams corresponding to different corrosion degrees, and the results of the analyses are presented in Figure 8-32 in terms of load–displacement curves. The plotted displacement corresponds to the vertical displacement at the midspan of the most heavily loaded external girder. The maximum value of the load multiplier, λ_{FEM} , represents the global capacity of the deck, where $\lambda_{FEM} = 1$ corresponds to the characteristic load as defined in [127].

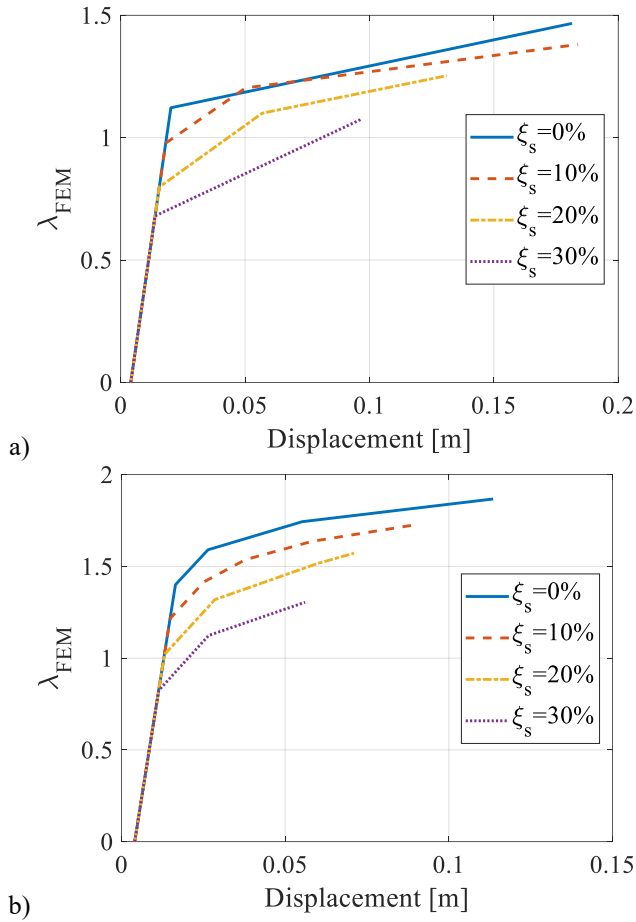


Figure 8-32: Load multiplier evaluated using the NLFE model for: a) tandem load placed at midspan; b) tandem load placed near the support.

λ_{FEM} ranges between 1.08 and 1.87 depending on the load configuration and the corrosion degree. Table 8-6 summarises the estimated values.

Table 8-6: Estimated load multipliers for both load cases at varying corrosion levels obtained using the NLFE model

Corrosion degree (ξ_s)	λ_{FEM} for tandem load at the midspan	λ_{FEM} for tandem load in proximity of the support
0%	1.47	1.87
10%	1.38	1.73
20%	1.25	1.57
30%	1.08	1.30

8.2.2.3 Comparison between the NLFE and the adopted brittle Daniels System estimated load multipliers

The load multipliers for the two load cases are evaluated using the brittle DS modelling proposed in the thesis. For both load configurations, the moment and shear demands are evaluated for the three main girders using the Courbon method, and for each section the ratio of capacity to demand is calculated. In this case, both capacity and demand for each section i are evaluated using the proposed brittle DS formulation:

$$C_{S,i} = \max\{2 \cdot \min[C_{1,i}, C_{2,i}], \max[C_{1,i}, C_{2,i}]\} \quad (92)$$

$$D_{S,i} = D_{1,i} + D_{2,i}$$

where $C_{S,i}$ is the system capacity, $D_{S,i}$ is the system demand due only to the traffic loads, $C_{1,i}$ is the capacity of the external girder, $C_{2,i}$ is the capacity of the internal girder, and $D_{1,i}$ and $D_{2,i}$ are the respective demands due only to the traffic loads. All these quantities are evaluated for the section i , and the load multiplier is defined as the minimum, over all sections, of the ratio between the available traffic load-carrying capacity, i.e. the system capacity reduced by the permanent load effects of both girders, $G_{1,i}$ and $G_{2,i}$, and the traffic load demand:

$$\lambda_{bDS} = \min_i \left\{ \frac{C_{S,i} - G_{1,i} - G_{2,i}}{D_{S,i}} \right\} \quad (93)$$

Table 8-7 shows the calculated load multipliers using the proposed bDS modelling.

Table 8-7: Estimated load multiplier for both load cases at varying corrosion degrees using the bDS modelling

Corrosion degree (ξ_s)	λ_{bDS} for tandem load at the midspan	λ_{bDS} for tandem load in proximity of the support
0%	1.26	1.63
10%	1.20	1.57
20%	1.07	1.40
30%	0.94	1.22

To compare the load multipliers evaluated using the NLFE and the bDS models, the relative error is computed for both load cases. The resulting values of ε_S are depicted in Figure 8-33:

$$\varepsilon_S = \frac{\lambda_{bDS} - \lambda_{FEM}}{\lambda_{FEM}} \cdot 100 \quad (94)$$

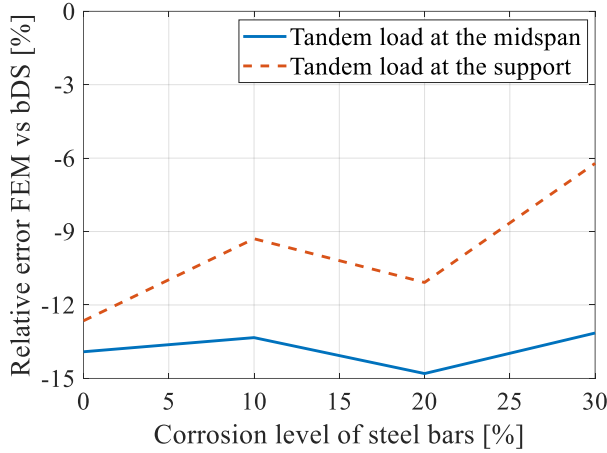


Figure 8-33: Relative error of the load multipliers evaluated using the brittle Daniels System model with respect to the NLFE model.

Again, Figure 8-33 shows that the simplified representation of the system behaviour through a brittle Daniels System is conservative, as the resulting load multipliers are smaller than those estimated using the NLFE model. For the first load case, where the tandem load is placed at midspan, the error ranges between -15% and -13% , depending on the corrosion level of the external girder. For the second load case, the error ranges between -13% and -6% . Moreover, the error is significantly influenced by the considered load configuration, confirming that the degree of conservativeness depends on the underlying redistribution mechanisms. Also in this case, the level of conservativeness can be considered moderate, indicating that the proposed DS modelling provides an approximate yet sufficiently accurate estimate of the system capacity.

Similarly to what is found in Section 8.1.1.3, plastic redistribution capacity of the deck depends on the load configuration, thus, for a spatial dependent analysis involving multiple potential failure modes and critical sections, several nonlinear probabilistic analyses accounting for a spatial time dependent random field modelling of the load would need to be performed, confirming the limitations found in the literature, for instance, [89], [95].

8.2.3 Limit State Functions

The equations provided in Chapter 4 are adapted to the bending, shear, and half-joint failure modes. In particular, Eq. (7) can be adapted as follows:

$$g_{M,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{M,C} M_{C,i}(\mathbf{X}, t) - \theta_{M,D} M_{D,i}(\mathbf{S}) \quad (95)$$

$$g_{V,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{V,C} V_{C,i}(\mathbf{X}, t) - \theta_{V,D,i} V_{D,i}(\mathbf{S}) \quad (96)$$

$$g_{HJ,i}(\mathbf{X}, \mathbf{S}, t) = \min[\theta_{V,C} N_{S,i}(\mathbf{X}, t) - \theta_{V,D} N_{D,S,i}(\mathbf{S}) ; \theta_{V,C} N_{T,i}(\mathbf{X}, t) - \theta_{V,D} N_{D,T,i}(\mathbf{S})] \quad (97)$$

In Eq. (95), $\theta_{M,C}$ represents the model uncertainties for the bending capacity model, $M_{C,i}(\mathbf{X}, t)$ represents the bending capacity as a function of the vector of capacity variables $\mathbf{X}$ and time t , at section i . $\theta_{M,D}$ is the model uncertainty for the bending demand model, and $M_{D,i}(\mathbf{S})$ represents the bending demand at section i which depends on the vector of demand variables $\mathbf{S}$, modelled here as time-independent. Similarly, in Eq. (96), $\theta_{V,C}$ represents the model uncertainties for the shear capacity model, $V_{C,i}(\mathbf{X}, t)$ denotes the shear capacity at the time t and section i . The shear demand model consists of $\theta_{V,D}$ for model uncertainties and $V_{D,i}(\mathbf{S})$ for the shear demand at section i . Finally, in Eq. (97), $N_{S,i}(\mathbf{X}, t)$ and $N_{D,S,i}(\mathbf{S})$ represent the axial capacities of the strut and tie, respectively, while $N_{T,i}(\mathbf{X}, t)$ and $N_{D,T,i}(\mathbf{S})$ represent their axial demands. All the variables are evaluated at section i .

8.2.3.1 Bending Capacity

The bending capacity $M_C(\mathbf{X}, t)$ of RC sections is based on the equilibrium of compressive and tensile forces at reinforcement yielding, in accordance with Eurocode provisions [128].

$$M_C(\mathbf{X}, t) = A_s(t) f_y \left[H - c_c - 0.5 \frac{A_s(t) f_y}{b_w f_c} \right] \quad (98)$$

where $A_s(t)$ is the area of the rebars [mm²], f_y is the steel yielding stress [MPa], H is the beam height [mm], b_w is the width of the beam [mm], f_c is the concrete compressive strength [MPa], and c_c is the concrete cover [mm].

8.2.3.2 Shear Capacity

The shear capacity $V_C(\mathbf{X}, t)$ is calculated using the modified compression field theory, the standard shear design model in Eurocode 2 [128]:

$$V_C(\mathbf{X}, t) = \min \left(\sum_{i=1}^{n_{st}} A_{sw,i}(t) \cdot f_{yw} ; 0.5 f_c \frac{\cot \theta}{1 + \cot^2 \theta} z b_w \right) \quad (99)$$

where f_c is the concrete compressive strength, z is the inner lever arm of the section, b_w is the section web width, θ is the angle of the compressive strut relative to the beam longitudinal axis, n_{st} is the number of stirrups intersected by the strut, $A_{sw,i}(t)$ represents the cross-sectional area of each stirrup (or inclined reinforcement) at time t , and f_{yw} is the yield strength of the stirrups or inclined reinforcements.

8.2.3.3 Half-joint connection Capacity

The axial capacities of struts and ties are calculated as [128]:

$$N_S(\mathbf{X}, t) = 0.6 \left(1 - \frac{f_{ck}}{250} \right) \cdot f_c \cdot b_w \cdot k_s ;$$

$$N_T(\mathbf{X}, t) = \sum_{u=1}^2 A_{s,u}(t) \cdot f_y \cdot \cos(\gamma)$$
(100)

where f_{ck} is the characteristic compressive resistance of the concrete, f_c is the concrete compressive strength, b_w is the effective width of the strut accounting for potential spalling, e.g., Eq. (100), k_s is the thickness of the strut, f_y is the steel yield strength, and γ is the inclination of the tie.

The design documentation and the strut-and-tie model used for their analysis are presented in Figure 8-34.

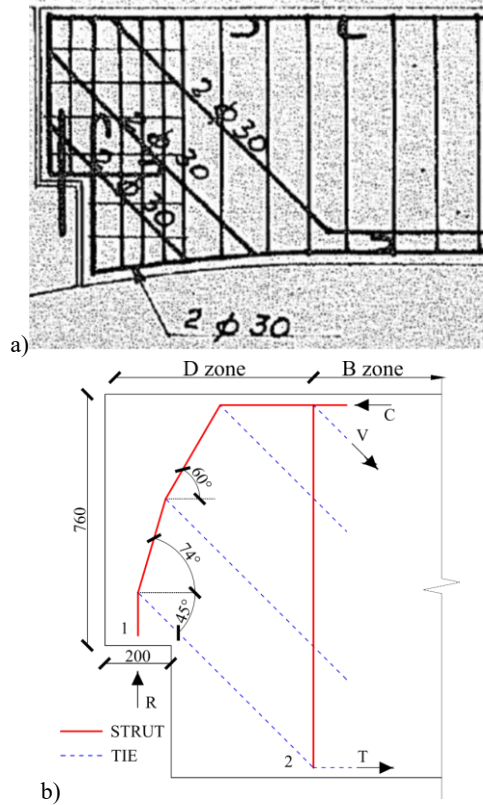


Figure 8-34: a) Half-joint design documentation; b) strut-and-tie model and critical elements (1: strut, 2: tie).

The strut-and-tie model is consistent with the original design documentation. For reliability analysis, only two critical elements are considered: (1) the strut with the smallest cross-section and highest compressive load, and (2) the most heavily loaded tie. In all formulations, spalling effects can be accounted for by replacing b_w with the effective width $b_{w,eff}$ as defined in Eqs. (51) and (52).

Eqs. from (95) to (100) are incorporated into Eqs. (1), (2), and (3) to define the LSFs for the girders and the deck under both major repair and global collapse scenarios.

8.2.3.4 Limit State Functions of the Crossbeams

The reliability of the crossbeams are evaluated to verify whether the crossbeams are able to transfer the load throughout the whole timespan analysed, as requested by the proposed framework.

To estimate the action on the crossbeams, the Courbon method is again employed. A section located at a distance equal to the crossbeam height from the external girder is assessed with respect to shear in the reliability analysis. The following time dependent LSF is defined, and Figure 8-35 illustrates the detailed reinforcement layout of the crossbeams:

$$g_{CB,i}(\mathbf{X}, \mathbf{S}, t) = \theta_{V,C} V_{C,CB,i}(\mathbf{X}, t) - \theta_{V,D} V_{D,CB,i}(\mathbf{S}) \quad (101)$$

where $V_{C,CB,i}(\mathbf{X}, t)$ is the shear capacity of the cross-beam as a function of the vector of capacity variables $\mathbf{X}$ and time t , $\theta_{V,C}$ and $\theta_{V,D}$ represent the model uncertainties for the shear capacity and demand models, and $V_{D,CB,i}(\mathbf{S})$ is the shear demand evaluated using the Courbon method depending on the vector of demand variables $\mathbf{S}$. Both $V_{C,CB,i}(\mathbf{X}, t)$ and $V_{D,CB,i}(\mathbf{X}, t)$ are referred to the i -th section analysed. The shear capacity can be evaluated using Eq. (86) as adopted for the main girders.

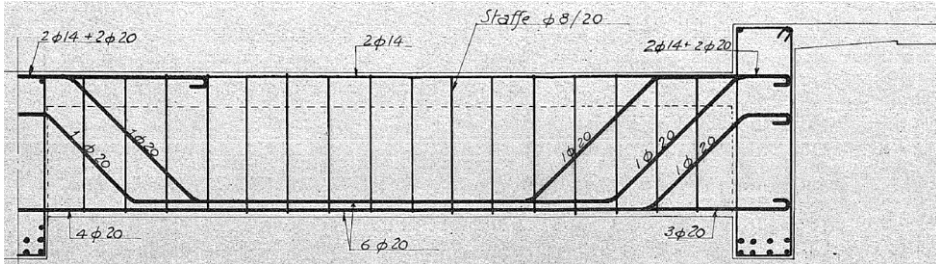


Figure 8-35: Excerpt of the original documentation showing the reinforcements details of the crossbeams

8.2.3.5 Load modelling

The permanent load per girder is estimated at 16.8 kN/m, while traffic loads are calculated using Load Model 1 from Eurocode 1 [127], Eq. (73), and [129]. Loads are placed on the deck to maximise effects on the external girder, as illustrated in Figure 8-36, which shows the midspan section of the half-joint deck. In accordance

with the Eurocode 1, the remaining area is not loaded, as its inclusion would reduce the demand on the external girder. Load effects are calculated with Euler-Bernoulli beam theory, with the Courbon method used to distribute loads transversely among girders.

The data collected through the WIM system are employed to develop a data-driven traffic load model, as described in [135]. Gumbel distributions of annual maximum effects are obtained, which are used to calibrate reduction factors for EC1 Load Model 1 [127]. The data-driven probabilistic traffic load model derived from WIM measurements is not directly used in the spatial reliability analysis, because it provides annual maximum distributions for specific response quantities (support shear and midspan bending moment) only. For the spatial analysis, the maximum shear and bending demand must be evaluated at each analysed section under critical load positioning. Therefore, the Gumbel distributions obtained from the WIM data are used to calibrate reduction factors for Eurocode 1 Load Model 1, while maintaining the load geometry as prescribed [127]. The obtained coefficient of variation ranges between 4% and 11% for spans comparable to the analysed bridge with an average of 8.6% depending on the considered effect (shear or bending) and the traffic stream, in agreement with the literature (see, for instance, [12]). The reduction factors are applied to the characteristic loads defined by EC1 Load Model 1 [127], consistently with the methodology proposed in [135]. The corresponding average coefficient of variation (8.6%) is then used to derive the parameters (mean and standard deviation) of the probabilistic models for the annual maximum traffic loads.

The resulting probabilistic models are reported in Table 8-8.

Table 8-8: Data-driven traffic load models.

Variable	Distribution	Mean	Standard Deviation	Reduction factor
Q_1 [kN]	G	105.3	9.06	$\beta_{Q_1} = 1$
q_1 [kN/m ²]	G	4.30	0.37	$\beta_{q_1} = 0.68$
Q_2 [kN]	G	16.8	1.44	$\beta_{Q_2} = 0.23$
q_2 [kN/m ²]	G	0.80	0.07	$\beta_{q_2} = 0.46$

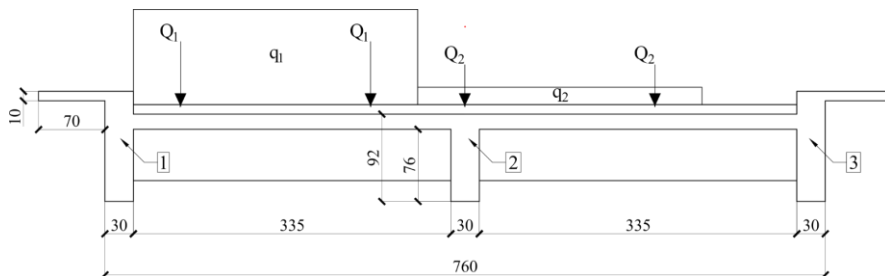


Figure 8-36: Arrangement of the loads on the deck.

8.2.4 Corrosion modelling

Both the longitudinal and transverse reinforcements (stirrups) are considered susceptible to corrosion. Generalised corrosion initiation is modelled using the carbonation model introduced in Section 7.1.1 (Eq. (34)), whereas pitting corrosion initiation follows Eq. (38). The complete corrosion activation model is presented in Chapter 7 (see Eqs. from (31) to (33)). These models are subsequently updated using the data acquired through the procedures introduced in Section 4.4, as illustrated below. Instead, corrosion propagation is modelled as presented in Chapter 7 (Eqs. from (47) to (50)).

8.2.4.1 Updating of the carbonation model

The best-fit distribution of the carbonation parameter, selected using the Cramér-Von Mises criterion [116], [117] is a Log-Normal distribution, as further confirmed by the probability plot shown in Figure 8-37.

This fitted Log-Normal distribution is adopted in Eq. (36) as the likelihood distribution for updating the carbonation parameter, while Eq. (37) is employed to estimate the posterior distribution in conjunction with the prior model $p'_k(k)$ of the carbonation parameter defined in Eq. (91). A comparison between prior, likelihood, and posterior distributions is depicted in Figure 8-38.

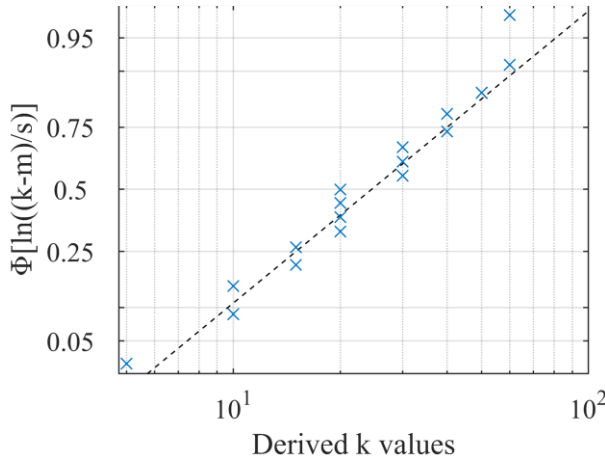


Figure 8-37: Probability plot of the Log-Normal distribution fitted to the carbonation parameters derived from test data.

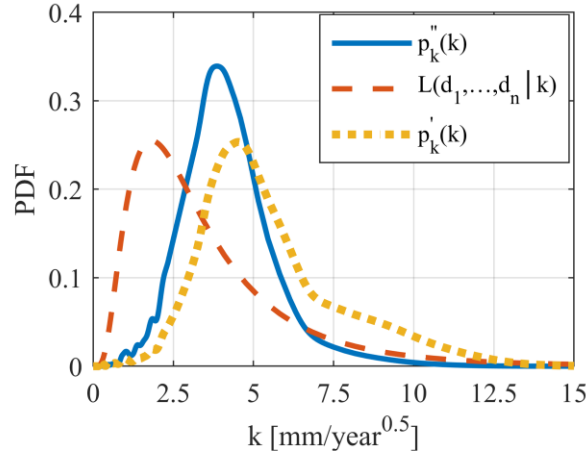


Figure 8-38: Comparison of prior, likelihood, and posterior distribution of the carbonation parameter

8.2.4.2 Updated chloride diffusion model

The method presented in Section 7.1.2.1 is applied to the most critical location, i.e., the position with the highest measured chloride content (cores D and E), as shown in Figure 8-39.

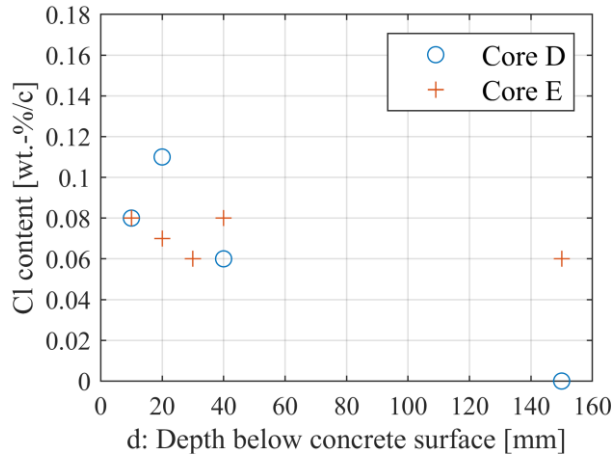


Figure 8-39: Scatter plot of chloride content at different depths for two concrete cores.

The non-linear model used in the regression is expressed as (see also Eq. (39)):

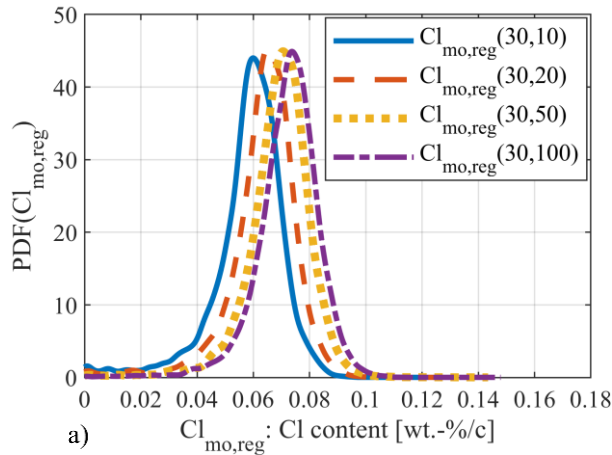
$$M_{ch}(\Theta_{ch}, d, 57) = 0 + (C_{s,app} - 0) \left[1 - \operatorname{erf} \left(\frac{d - 0}{2\sqrt{D_{app}(57) \cdot 57}} \right) \right] \quad (102)$$

The term $D_{app,c}(57)$ represents the apparent chloride diffusion coefficient evaluated at year 57. Following literature recommendations [67], the depth of the convection zone is assumed to be 0 mm and is not further estimated. Eqs. (42) and (43) are employed to estimate the parameters, their squared errors, and the correlation coefficient. The results are shown in Table 8-9.

Eqs. (43) and (44), evaluated in year 57, can then be used to calculate the chloride concentration at the cover depth for different service years. Figure 8-40a shows chloride concentrations estimated at 10, 20, 50, and 100 years at a cover depth of 30 mm using the data-driven model. For comparison, the corresponding prior model estimates (using the prior models for the variables shown in Table 8-11) are shown in Figure 8-40b.

Table 8-9: Estimated parameters (expected value, squared error, and correlation coefficient).

Parameter	Expected value	Squared error	Correlation coefficient
$C_{s,app,data}$ [wt.-%/c]	0,0868	0,0106	-0,54
$D_{app}(57)$ [mm ² /year]	207,97	162,39	



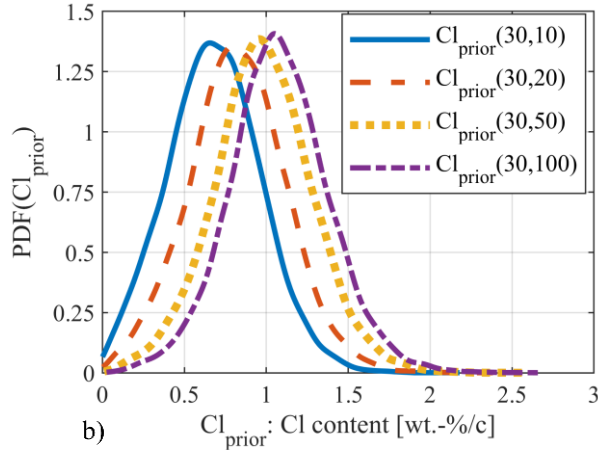


Figure 8-40: Chloride content at cover depth estimated for 10, 20, 50, and 100 years using a) the data-driven model; b) the prior model.

Figure 8-40 shows that the data-driven model (Eq. (40)) captures the time-dependent behaviour characterising the prior model. In both cases, the expected chloride concentration increases with time, and the uncertainty in predictions grows with longer forecast horizons, as expected. However, the predictions from the calibrated model have a lower coefficient of variation than those from the prior model, reflecting reduced uncertainty in chloride diffusion.

The distributions of both the apparent surface chloride concentration ($C_{s,app}$) and the chloride diffusion coefficient ($D_{RCM,0}$) are updated using a Bayesian framework. The prior distributions are given in Table 8-11, while the likelihood distribution for $C_{s,app}$ is modelled as Normal with mean 0.0806 and standard deviation 0.0106. The likelihood for $D_{RCM,0}$ is derived from Eq. (43), where $D_{app}(57) \sim N(207,97; 162,39)$. Posterior distributions are then obtained using Eq. (14). In particular, the posterior distribution of $C_{s,app}$, $p''_{C_{s,app}}$, (Figure 8-41) is narrower and shifted toward lower values compared to the prior, indicating a lower probability of pitting corrosion for the assessed structure. The predictive distribution of T_p is evaluated using Eq. (15).

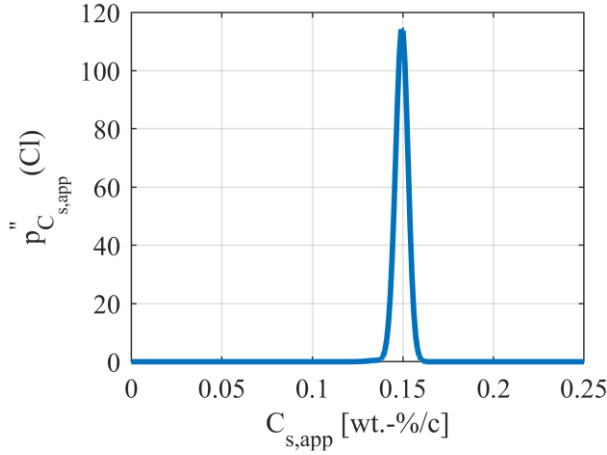


Figure 8-41: Marginal posterior distribution for the chloride content at the concrete surface.

8.2.4.3 Correlation modelling

RFs are employed to model the correlation of spatially dependent variables [29]. A Gaussian correlation function with a correlation length of 2 m is adopted for all variables, as recommended in [30]. The RFs used are summarised in Table 8-10. The marginal distribution of the RF associated with $C_{s,app}$ and $D_{RCM,0}$ are updated as described in Section 8.2.2.2, while the concrete compressive strength distribution is updated using its conjugate distribution as likelihood, resulting in a Log-Student t (Log- t) predictive distribution. In all cases, only the marginal distributions are updated, while the correlation structure is kept as in the prior.

The yield stress of longitudinal reinforcement is modelled as fully correlated within the same girder and partially correlated between girders $\rho_{f_y} = 0.4$ [99]. The yield stress of stirrups is assumed to be partially correlated across all stirrups, with a correlation coefficient $\rho_{f_{yw}} = 0.4$ [99]. Finally, the prior probabilistic models used in the analysis are presented in Table 8-11, while the updated variables are listed in Table 8-12.

Table 8-10: Variables modelled as RFs to account for spatial correlation

Variable and reference	RF distribution	Mean	Standard Deviation	Correlation type	Correlation length [m]
C_{cr_c} [30]	B [0.2; 2]	0,6	0,15	Gaussian	2
C_{cr_s} [30]	LN [0.2; 2]	1,25	0,37	Gaussian	2
$C_{s,app}$ [30]	NP	0,17	0,004	Gaussian	2
$D_{RCM,0}$ [30]	NP	500.4	101.59	Gaussian	2
f_c [30]	Log-t	31,59	9,34	Gaussian	2

Table 8-11: Prior distribution of the input variables.

Variable	Distribution	Mean	Standard Deviation	Reference
f_c [MPa]	LN	28,28	5,92	[136]
EC	D	2	-	-
RH [%]	W [0;100]	75	15	[65]
$[CO_2]$ [%]	D	0,04	-	[134]
C_l [kg/m ³]	U [275; 295]	285	-	-
k [mm ² /year ^{1/2}]	NP	5,52	2,18	-
c_c [mm]*	D	30 or 90 or 150	-	-
c_w [mm]	D	30	-	-
$\mu_{C_{s,Ad}}$ [wt.-%/c]	U [1; 2]	1.5	0.29	[67]
$C_{s,app}$ [wt.-%/c]	NP	μCs	$\mu Cs * 0.45$	[67]
C_{cr_c} [wt.-%/c]**	B [0.2; 2]	0,6	0,15	[67]
C_{cr_s} [wt.-%/c]**	LN [0.2; 2]	1,25	0,37	[132]
Δd [mm]	D	0	-	[67]
$D_{RCM,0}$ [mm ² /year]	N	498,27	99,65	[67]
b_w [mm] ***	D	300 or 450	-	-
K_c	N	1	0,1	[52]
α (pitting factor)	LN	9,28	4,04	[67]
$V_{corr,a}$ [mm/year]	W	0,03	0,02	[65]
ToW [65]	N	0,75	0,2	[65]
f_y, f_{yw} [MPa]	N	322,34	26,59	[137]
$\theta_{M,C}$	LN	1,2	0,15	[99]
$\theta_{M,D}$	LN	1	0,1	[99]
$\theta_{V,C}$	LN	1	0,1	[99]
$\theta_{V,D}$	LN	1	0,1	[99]
k_s [mm]	D	150	-	-

Application of the Framework to Case Studies

ϕ_w [mm]	D	8	-	-
D_0 [mm]	D	30	-	-
q_G [kNm] *****	N	16,85	0,84	-
Q_1 [kN] *****	G	115.9	6.94	[127]
q_1 [kN/m2] *****	G	6.95	0.42	[127]
Q_2 [kN] *****	G	77.2	4.63	[127]
q_2 [kN/m2] *****	G	1.93	0.12	[127]

N: Normal; D: Deterministic; LN: Log-normal; W: Weibull; B: Beta; U: Uniform; NP: Non parametric; G: Gumbel. *30, 90 or 150 for the first, second, or third layer of longitudinal reinforcements, respectively. **Depending on whether concrete is carbonated or not. *** 450 at the half joints, 300 otherwise. **** Evaluated considering a coefficient of variation equal to 0.05. ***** Evaluated considering a coefficient of variation of the EC1 [127] load scheme 1 equal to 0.06.

Table 8-12: Posterior distribution of the updated variables.

Variable	Distribution	Mean	Standard Deviation
f_c [MPa]	Log-t	31,59	9,34
k [mm ² /year ^{1/2}]	NP	4,24	1,44
$C_{s,app}$ [wt.-%/c]	NP	0,17	0,004
$D_{RCM,0}$ [mm ² /year]	NP	500,4	101,59
Q_1 [kN]	G	105.3	9.06
q_1 [kN/m ²]	G	4.30	0.37
Q_2 [kN]	G	16.8	1.44
q_2 [kN/m ²]	G	0.80	0.07

8.2.5 Results and discussion

The crossbeams reliability is assessed and compared with the reliability of the main girders. The crossbeam time-dependent reliability index ($\beta_{Crossbeam}$) is shown together with the reliability index of the external main girder ($\beta_{Girder1}$) in Figure 8-44.

Figure 8-42 shows that the reliability index of the crossbeams is consistently higher than that of the main girder throughout the entire analysis period. Consequently, the probability of failure of a crossbeam remains more than an order of magnitude lower than that of a girder at any time.

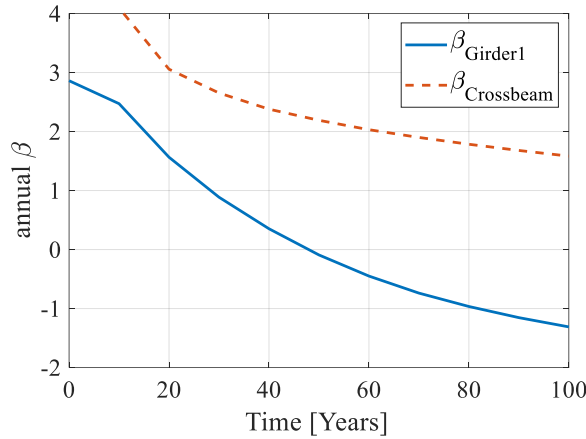


Figure 8-42: Time dependent reliability index of the crossbeams and of Girder 1

Similarly, the series system reliability is evaluated under two configurations: (i) considering a series system composed of both girders and crossbeams; and (ii) considering a series system composed only of the girders. Correlation among components is modelled using random fields for the degradation-related variables and for the concrete compressive strength, while model uncertainty variables and traffic loads are assumed to be fully correlated, as discussed in Section 8.2.3. The results are shown in Figure 8-43.

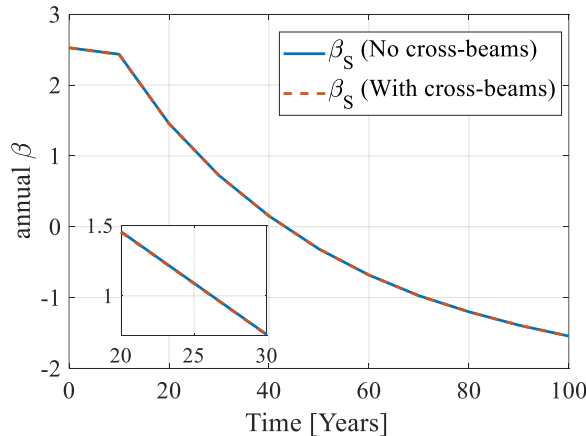


Figure 8-43: System reliability with or without accounting for crossbeam failure

Figure 8-43 compares the annual system reliability index evaluated without accounting for crossbeam failure and that obtained when crossbeam failure is

included as an additional system component. The figure shows that the estimated annual reliability is virtually unchanged throughout the entire analysis period.

Therefore, the crossbeams can be considered to retain adequate reliability for transferring loads between the girders over the analysed service life.

This result supports the modelling assumption adopted within the brittle DS approach. Since the crossbeams exhibit reliability indices significantly higher than those of the longitudinal load-carrying members, the governing system failure is controlled by the longitudinal girder behaviour. Accordingly, neglecting crossbeam failure (i.e., treating crossbeams as non-governing components in the system formulation) does not introduce a bias in the probabilistic assessment. Together with the NLFE comparisons (Section 8.2.1.3), this indicates that the combined Daniels/Courbon approximation is computationally efficient and only moderately conservative (~ -6 to -15%) for the studied RC deck.

The results of this verification step allow using the proposed framework for the estimation of the spatial and time-dependent reliability of the deck.

Figure 8-44 shows the prior and posterior annual reliability indices of the deck, and the indices accounting for survival information, for both the series system model ($\beta_S, \beta_{S,I}, \beta_{S,post}$ and $\beta_{S,I,post}$) and the brittle DS model ($\beta_{bDS}, \beta_{bDS,I}, \beta_{bDS,post}$ and $\beta_{bDS,I,post}$). Compared with the prior analysis, incorporating updated information increases the initial reliability and slows down corrosion progression, resulting in an extended technical service life. Indeed, the estimated service life before major repair (series system model) is 77 years, representing an 17-year extension compared with the nominal fifty-year service life. For structural collapse (brittle DS), the service life is 79 years, a 19-year extension. The estimated service life is thus longer than the fifty-year service life typically considered for bridges built in Italy during the sixties. Moreover, these results confirm the value of incorporating monitoring and test data into service-life assessments. By extending the predicted service life, intrusive repairs can be delayed or avoided, enabling more cost-effective and sustainable maintenance strategies.

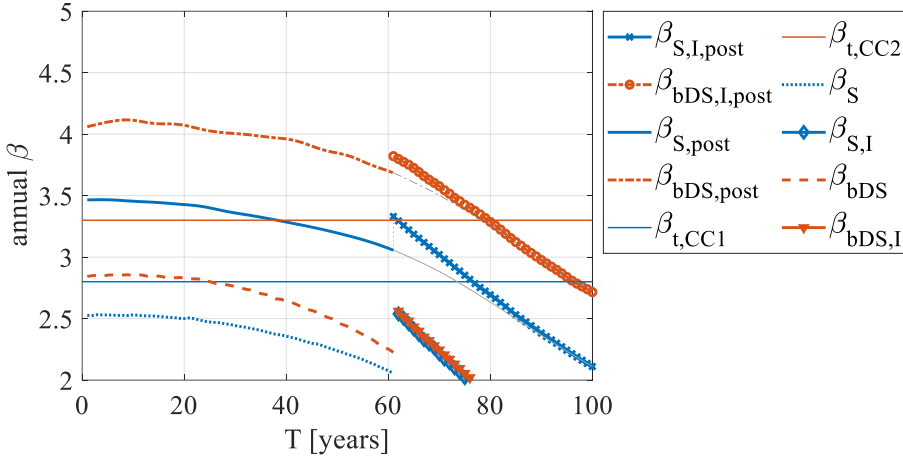


Figure 8-44: Prior and posterior global annual reliability of the deck for series and brittle Daniels System modelling.

8.2.5.1 Sensitivity analysis

A sensitivity analysis is conducted to identify the most influential variables, i.e., those for which reducing uncertainty has the most significant effect on reliability and service life. The analysis is performed for both the prior and posterior cases. Spearman correlation coefficients (ρ) are calculated for each input variable relative to the LSF. Figure 8-45 shows the results for the most loaded girder in the prior case, considering all failure modes (shear, bending, and half-joint). For clarity, only the twelve most sensitive variables are presented.

Figure 8-45 reveals that shear capacity model uncertainty ($\theta_{V,C}$) and the yield resistance of the longitudinal reinforcements (f_y) dominate in the low-degradation state of the bridge. This is expected, as f_y influences all three failure modes due to the inclined reinforcement layout (see Figure 8-34). On the Demand side, shear demand model uncertainty ($\theta_{V,D}$) and both distributed (q_1) and concentrated (Q_1) traffic loads are initially the most sensitive variables.

As degradation progresses, corrosion-related variables ($\alpha, ToW, V_{corr,a}, T_{G|P}$) become increasingly important. In particular, the initiation time of corrosion $T_{G|P}$ and the pitting factor α become dominant after about 35 years, highlighting the benefit of reducing uncertainties in these parameters to improve reliability estimates.

The posterior sensitivity analysis (Figure 8-46) shows important differences. While the same variables ($\theta_{V,C}, f_y, \theta_{V,D}, Q_1, q_1$) remain dominant, as the dominant failure modes are half-joint failure in the initial stage and shear in the later stage of the analysis, the influence of degradation-related variables is markedly reduced. In

particular, $T_{G|P}$, α , ToW , and $V_{corr,a}$ show sensitivities close to zero. This shift reflects the Bayesian updating of the corrosion process: given the very low measured chloride content, generalised corrosion now governs deterioration rather than pitting.

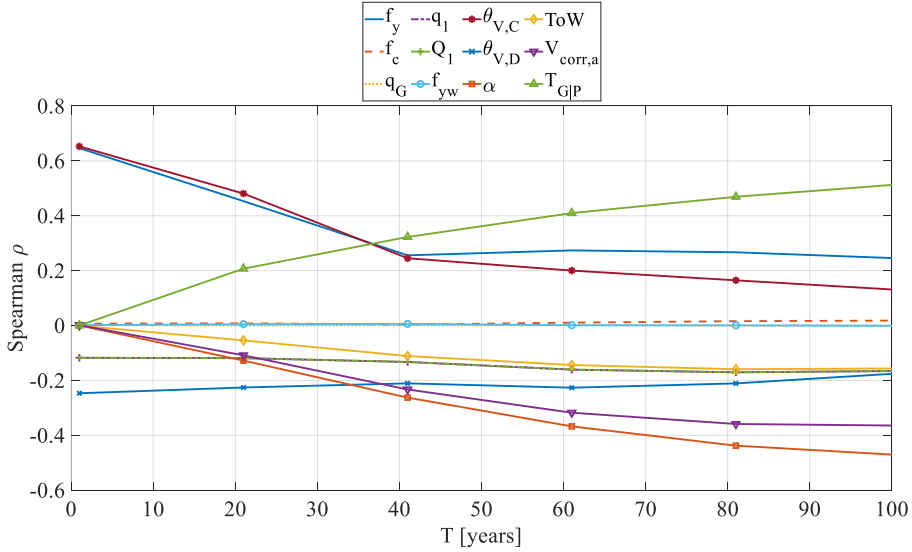


Figure 8-45: Sensitivities of the LSF in the prior case considering all failure modes for the most loaded girder.

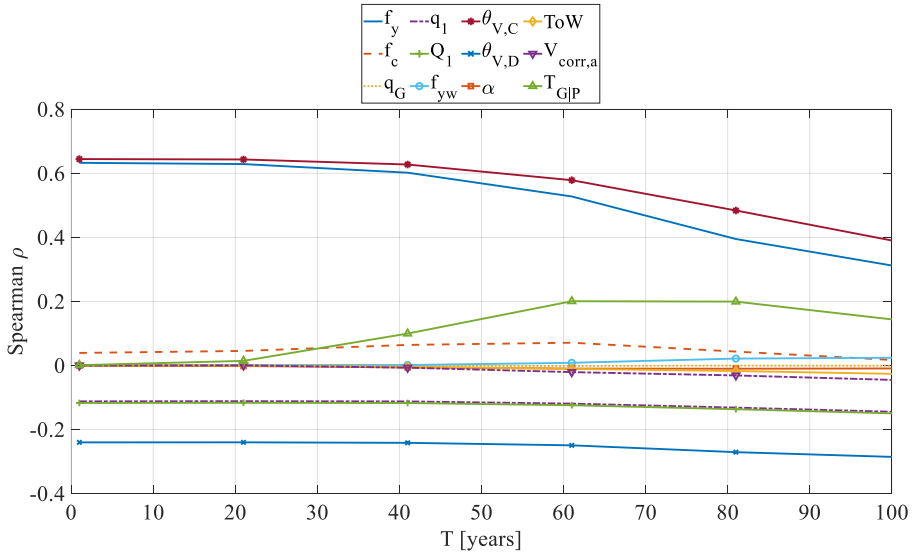


Figure 8-46: Sensitivities of the LSF in the posterior case considering the three failure modes for the most loaded girder.

Notably, the posterior analysis highlights two key parameters: concrete compressive strength (f_c) and the stirrup yield strength (f_{yw}). This can be explained by the dominance of half-joint failure in the early stages (sensitive to f_c) and of shear failure in the later stages, after year 65, (sensitive to f_{yw}). In particular, f_{yw} shows a sharp increase in sensitivity between years 60 and 80, reflecting its crucial role in shear capacity as degradation advances.

8.2.5.2 Influence of the spatial modelling on the reliability calculations

The effect of correlation length on reliability estimates is tested by varying the correlation length of spatially dependent variables from 1 m (lower bound from the literature [30]) to 3.5 m (upper bound [30]), compared with the baseline value of 2 m. This analysis also illustrates the potential gains in service-life estimation from updating not only marginal distributions but also the correlation structure of random fields.

The resulting posterior global annual reliability indices ($\beta_{S,1}$, $\beta_{S,3.5}$, $\beta_{bDS,1}$, and $\beta_{bDS,3.5}$) are compared with those obtained using a correlation length of 2 meters ($\beta_{S,2}$ and $\beta_{bDS,2}$). The results are illustrated in Figure 8-47.

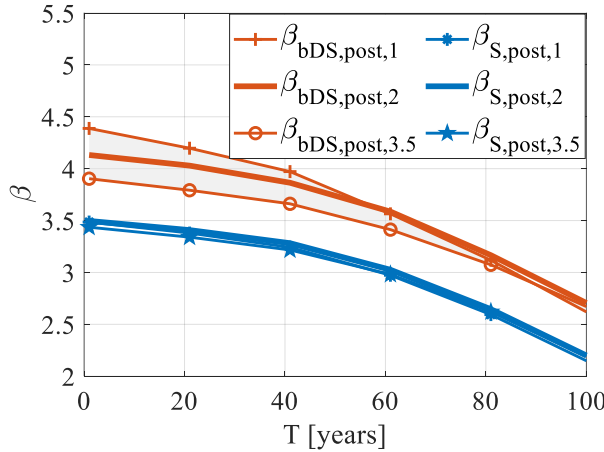


Figure 8-47: Annual reliability index with error bands for different correlation lengths of RFs.

Figure 8-47 demonstrates that system reliability results are only marginally affected by the choice of correlation length, confirming that a value of 2 m is robust and appropriate [30]. For the series system ($\beta_{S,post}$), differences remain negligible, with reliability varying by less than 2% at 3.5 m. With a 1 m correlation length, initial reliability is about the same as the 2 m case, but as corrosion progresses, reliability exceeds that in the 2 m case, with a maximum difference of 2%. This is because

corrosion development reduces correlation among components over time, and shorter correlation lengths result in lower average corrosion levels.

For the brittle DS model, the influence of correlation length is more pronounced in the early years. The largest difference ($\approx 6\%$) is observed in sound conditions, mainly due to higher correlation among half-joint sections. In brittle DS systems, lower correlation reduces the likelihood of simultaneous failures, leading to slightly higher reliability [19]. The differences in the three cases diminish as corrosion progresses, and component correlation naturally decreases.

8.3 Summary of the Chapter

The System Reliability Framework is applied to both a PT and an RC bridge deck, demonstrating its capability for system reliability assessment and service-life estimation across different structural typologies. For the PT deck, the tendon degradation model developed in Chapter 7 is implemented to represent tendon damage under a system perspective, enabling the estimation of residual service life beyond deterministic assumptions. For the RC deck, information gathered from multiple sources, material testing, monitoring, and survival evidence is integrated into the framework, allowing verification of the structure's reliability that would not have been possible based solely on prior models. Moreover, for both case studies, the approximation introduced by the brittle Daniels System idealisation of the system capacity has been evaluated. In both cases the Daniels system formulation results in a slightly conservative estimate (approximately -8 to -18%), showing its suitability for the two case studies analysed.

These applications underline two complementary properties of the framework. In the PT case, the approach captures the influence of defect occurrence and spatially variable corrosion, demonstrating how probabilistic modelling can extend service-life estimates and guide more efficient maintenance planning. In the RC case, the framework demonstrates its flexibility in incorporating monitoring data, Bayesian updating, and posterior analyses, thereby reducing epistemic uncertainty and redirecting inspection priorities towards the parameters most relevant for the observed deterioration mechanisms.

The sensitivity analyses further highlighted the dependence of system reliability on the structural degradation level. They suggest that inspection campaigns should be adapted to the condition of the bridge: focusing on peak-load sections in sound or lightly degraded bridges and progressively extending to the entire structural element as degradation spreads. The spatial analysis of failure positions reinforced this conclusion, showing that sections far from peak-effect locations may govern failure once degradation is widespread.

Finally, for the RC case study, assumptions regarding spatial modelling based only on prior knowledge are shown to have a minor influence on system reliability and service-life estimation, confirming the robustness of the adopted correlation structure.

9 Conclusions

This thesis presents a comprehensive system reliability and updating framework for the service-life assessment of concrete bridges affected by corrosion. The framework integrates probabilistic modelling and in situ data from inspections, testing, and monitoring, enabling the formulation of probabilistic models that reflect actual degradation processes. Bayesian updating procedures are implemented to incorporate these data, improving the relevance of reliability calculations and service-life predictions. Corrosion degradation is modelled using a logical system approach that explicitly accounts for the influence of concrete quality on corrosion initiation.

The research work makes a conceptual contribution to the field of structural reliability. The framework advances existing approaches by explicitly combining: (i) a nested system representation that integrates multiple interacting failure modes, (ii) probabilistic modelling of spatially and temporally dependent degradation, and (iii) Bayesian updating procedures that preserve the temporal characteristics of deterioration processes. While previous studies have typically focused on isolated components or assumed uniform degradation, the proposed framework unifies these aspects into a coherent and computationally tractable methodology. It allows bridges to be modelled as nested systems of failure modes, sections, and elements (including both main girders and crossbeams), capturing interactions among degradation mechanisms and identifying the governing factors controlling reliability at different stages. Moreover, the mechanical model approximation introduced by modelling the grillage deck as a brittle Daniels System is explicitly quantified through comparison with nonlinear finite element models with lumped plasticity, considering two representative load configurations and multiple corrosion levels.

The generality of the developed framework ensures adaptability to both reinforced concrete and post-tensioned typologies, incorporates heterogeneous data sources, and provides service-life estimates directly relevant to management decisions. In particular, the framework and its complete application demonstrate how spatial degradation patterns, load positioning, and failure-mode interactions jointly influence system reliability, providing insights into and enhancing the service-life management of ageing bridge infrastructure.

Modelling of degrading structural systems is extended by coupling logical system representations with probabilistic formulations of corrosion initiation and propagation. Concrete quality and conservation state are integrated through logical combinations of carbonation and chloride ingress, allowing corrosion to be represented under different environmental and material conditions while preserving physical interpretability. These models enable consistent estimation of residual life within a probabilistic reliability framework.

The Bayesian updating procedures integrates both direct and indirect measurable variables, as well as evidence from probability-of-detection curves, and inequality and equality information. The developed procedures preserve the temporal characteristics of deterioration, particularly in the chloride ingress updating procedure, and enable the integration of heterogeneous data sources into the reliability assessment. As a result, service-life predictions become progressively more representative and relevant as new information is collected.

For post-tensioned bridges, the development of a dedicated inspection procedure enables the acquisition of detailed data on tendon degradation. The combined use of ultrasonic tomography and endoscopy facilitates the detection and characterisation of voids and corrosion within internal ducts. This procedure not only serves the immediate purpose of assessing tendon conditions but also generates a comprehensive dataset for Italian post-tensioned bridges, which is subsequently used to refine reliability-based assessment procedures by incorporating the actual condition of tendons.

The collected inspection data facilitated the development of probabilistic models describing the occurrence and spatial distribution of injection defects and corrosion in post-tensioned girders. These models explicitly account for detection uncertainties through probability-of-detection curves developed for the employed ultrasonic technique, ensuring consistency between observed and modelled defect frequencies. Built upon the extensive inspection database, the models are integrated into the time-variant system reliability framework to represent spatially dependent degradation processes and for use within Bayesian updating procedures.

An advanced structural reliability analysis scheme is established to perform spatially dependent system reliability analyses employing the developed models. In this procedure, both the number and positions of degrading sections are treated as random variables and simulated using the developed probabilistic models, overcoming the limitations of traditional segmentation techniques, whose results depend on arbitrary discretisation choices. The results show that failure tends to occur at later stages of the service life not only at the most highly utilised sections but also progressively throughout the beam.

The applicability and generality of the framework are demonstrated through two case studies on a post-tensioned and a reinforced concrete bridge deck. The comparative application confirms that the proposed approaches, models, and

procedures consistently integrate degradation models, inspection and monitoring data, and spatial variability into reliability assessments.

For both case studies, the mechanical model of the brittle Daniels System representation proved to be moderately conservative in the comparison to nonlinear finite element analyses (see Sections 8.1.1 and 8.2.2). A more relevant comparison would involve experimentally measured global capacities of bridge decks. The literature shows that nonlinear finite element models can be calibrated to accurately represent experimental results from full-scale failure tests. However, significant challenges arise when dealing with existing degraded decks. In such cases, failure tests can rarely be carried out, and the level of structural degradation cannot be fully characterised, making it difficult to calibrate and validate mechanical models, whether nonlinear or based on the Daniels System representation. This limitation highlights a niche to explore for reducing uncertainties in global capacity evaluation and improving the calibration of mechanical models for existing degraded bridge decks.

The nonlinear and brittle Daniels System mechanical model comparison shows that, the underestimation of the global system capacity ranges between 8% and 18% for the post-tensioned deck and between 6% and 15% for the reinforced concrete deck, depending on the load configuration and corrosion level considered. In particular, the deviation between the two modelling strategies decreases as degradation progresses, indicating that the brittle Daniels System provides increasingly comparable estimates of system capacity at higher degradation levels. This is particularly relevant because the Daniels mechanical representation requires substantially lower computational effort than nonlinear probabilistic finite element analyses. Notably, the system capacity is found to depend not only on the degree of corrosion but also on the load configuration, highlighting that, when spatial degradation and multiple potential failure sections are considered, nonlinear probabilistic finite element models would need to be developed together with spatially and temporally consistent probabilistic load modelling to fully capture redistribution effects.

These outlined aspects are currently a challenge as the computational burden associated with performing such nonlinear analyses within a probabilistic, time-variant framework is currently unsustainable for practical applications. In addition, maintaining consistent spatial and temporal correlation structures over time, essential for both Bayesian updating and integration of survival information, poses further methodological challenges. Therefore, the adoption of the analytically closed-form Daniels System mechanical model representation provides a compromise between mechanical realism and computational feasibility, enabling spatially and temporally consistent system reliability analysis within the probabilistic framework.

Time-dependent sensitivity analyses conducted for both reinforced and post-tensioned bridge systems support the identification of efficient inspection strategies to achieve target reliability levels, thereby promoting efficient condition assessment. These analyses reveal how the influence and uncertainty of different parameters evolve throughout the service life, supporting a rational prioritisation of inspection variables and measurement types that most effectively affect the structural reliability at each degradation stage. For post-tensioned bridges, the results demonstrate the evolution of system performance with respect to the number and spatial distribution of degrading sections, enabling the design of progressive inspection strategies that adapt as deterioration propagates. At the initial stage (low degradation levels), reliability is primarily influenced by model uncertainties and the mechanical properties of the materials. As degradation progresses, the time to corrosion activation, corrosion rate, and occurrence rate of corrosion become increasingly significant, as they determine both the extent and number of corroding sections.

Similarly, for reinforced concrete bridges, the analyses show that at low degradation levels, reliability is mainly governed by model uncertainties and material mechanical characteristics, whereas during advanced stages, the time to corrosion initiation, corrosion rate, and pitting factor become dominant parameters. However, as new data are incorporated through updating, the influence of corrosion-related variables decreases across the entire analysis timespan due to the reduction of their associated uncertainties, while model uncertainties remain the prevailing source of variability throughout the service life.

From a management perspective, the research has immediate practical implications. The use of survival information can significantly extend service life (by more than 10 years in the case studies), allowing managers to postpone interventions and reduce costs safely. Sensitivity analyses identify critical variables to monitor at different degradation stages, guiding inspection priorities and their evolution over time. Posterior sensitivity analyses show that degradation variables lose influence once measured, shifting attention to material strength parameters and model uncertainties. Moreover, the condition of Italian post-tensioned concrete bridges has been further clarified through the analysis of the case study. Despite design and construction flaws, such as injection voids, the results indicate an expected service-life extension of five years with respect to the assumed fifty-years service life. Nevertheless, these service-life estimations should be further substantiated using bridge-specific data.

This outcome highlights that utilising the developed data-based spatial degradation model results in service-life extension. For Italian infrastructure managers, this implies (i) the possibility of systematically assessing the service life of the post-tensioned bridge stock through representative case and parametric studies; and (ii) the possibility of performing bridge-specific service-life assessments in special cases (e.g., strategic infrastructures) by integrating structure-specific data.

Spatial failure analyses further reveal that inspection strategies should evolve with degradation, focusing initially on peak-load areas and progressively expanding as deterioration spreads. This behaviour is consistent with the observed sensitivities to the number and location of degrading sections. Moreover, variations in spatial correlation length are shown to have only a limited impact on global reliability estimates, reducing the urgency of refining correlation parameters and allowing resources to be prioritised elsewhere.

These findings demonstrate how the proposed framework can support more targeted inspections, cost-effective maintenance, and may lead to extended service life for post-tensioned and reinforced concrete bridges, all while maintaining safety consistent with international reliability requirements. By combining sensitivity analysis with Bayesian updating, the integration of inspections, tests, and monitoring data not only reduces epistemic uncertainty but also redirects inspection and maintenance priorities towards parameters that most directly affect structural reliability. This underlines the practical value of the framework in supporting rational, data-informed decision-making for ageing concrete bridges.

Beyond technical and managerial outcomes, the work has clear societal relevance, and its implementation within infrastructure management may generate a three-fold societal impact: (i) greater economic efficiency, (ii) enhanced sustainability, and (iii) improved life-safety.

Integrating the reliability calculations provided and the sensitivity analysis outcomes can achieve higher economic efficiency. Indeed, infrastructure managers and owners can perform maintenance only when needed, guided by rational prioritisation and intervention planning. This enables the efficient allocation of public and private resources while operating within financial constraints. In addition, resource allocation and long-term infrastructure planning contribute to resilient and accountable governance.

Moreover, extending the service life of existing infrastructure contributes to sustainability by reducing material consumption and waste generation, and by promoting more efficient resource use. This is aligned with the United Nations Sustainable Development Goals set out in the 2030 Agenda for Sustainable Development [138]. In particular, Figure 9-1 illustrates the relevant Sustainable Development Goals, highlighting those pursued throughout this thesis.

A deeper understanding of structural performance over time can also improve life-safety by reducing the risk of unexpected failures. Beyond direct safety benefits, the framework can strengthen public trust in infrastructure systems by demonstrating that decisions on monitoring and maintenance are grounded in quantitative, transparent assessments.



Figure 9-1: Significance of the Ph.D. thesis in the context of United Nations Sustainable Development Goals.

Finally, the thesis aligns with Pillar II - Cluster 3 of Horizon Europe (*Global Challenges & European Industrial Competitiveness, Civil Security for Society*), potentially delivering savings in infrastructure management while preserving low human-safety risks and ensuring equitable access to reliable transport networks.

Nevertheless, several limitations remain. The representation of corrosion processes, while grounded in empirical data, remains simplified and may be enhanced through more detailed physical-chemical modelling. The probabilistic model for post-tensioned girder degradation could be refined by expanding the inspection database to include a broader range of geometries and tendon configurations. The reliance on crude Monte Carlo simulations increases computational cost, while the use of advanced sampling and surrogate modelling techniques could achieve computational efficiency without compromising accuracy.

Given that the system capacity is evaluated conservatively within the adopted modelling approach, further refinements of the mechanical model, addressing computational burden and temporal correlation challenges, could enable more relevant reliability estimates. Such improvements may support additional service life extensions and enhance the socio-economic and sustainability impact of the research.

Finally, the study aims to provide a comprehensive assessment of structural reliability, and it could be further extended to incorporate maintenance planning and decision-analysis concepts, integrating reliability results into long-term asset management planning.

10 Future Works

The primary objective of this Ph.D. project is to address the problem of assessment and management of existing concrete bridges by developing a framework for calculating system reliability and estimating the service life of concrete bridges, explicitly integrating corrosion-induced degradation and the use of data gathered through in situ campaigns to enhance the relevance and accuracy of the calculations. Potential future research work, aimed at improving and expanding the proposed framework, is provided below. These directions encompass experimental investigations, real-world infrastructure management, and non-stationary environmental effects from climate change.

- **Experimental investigation:** the results obtained in this research demonstrate that the integration of inspection, testing, and monitoring data can be crucial for extending the service life of existing concrete bridges. However, the corrosion degradation models developed and applied in this Ph.D. work, although scientifically grounded and supported by inspection data, remain simplified representations of complex chemical and physical phenomena. Dedicated experimental studies on the interaction among different corrosion mechanisms and their boundary conditions are therefore recommended to improve understanding of degradation processes in existing concrete structures. Regarding PT tendon degradation, the proposed model currently considers only the spatial distribution of potential corrosion sites. Future studies could explore specific mechanisms governing corrosion initiation and propagation within ducts, thereby refining estimates of service life and improving predictiveness.
- **Decision support for management of infrastructure:** this work provides a rational basis and time horizon for prioritisation, inspection, monitoring, and retrofitting planning, achieved by estimating the remaining time before a structure exceeds its target reliability threshold. Although this is a useful advancement, the framework could be further enhanced by integrating risk management principles and decision-analysis methodologies. In terms of risk management, new target reliability levels could be defined based on specific risk considerations, including both direct and indirect consequences, thus providing managers with more precise guidance on the acceptable risk levels for individual bridges.

From a decision-analysis perspective, the framework could be expanded to support rational decision-making regarding test execution or monitoring, including the selection of appropriate instrumentation and the evaluation of alternative interventions. This would enable utility-based optimisation of management actions, such as deciding whether to retrofit, rebuild, or reduce loads, with both safety and cost-effectiveness considered.

- **Integration of non-stationary processes:** the degradation models utilised for both corrosion activation and propagation depend on environmental variables such as relative humidity, temperature, and atmospheric carbon dioxide concentration.

In this work, these variables are modelled as stationary; however, their non-stationary nature under climate-change scenarios may be incorporated in future analyses. Including time-varying environmental trends would enable more relevant predictions over longer time spans (e.g., when applied to a newly designed structure), as these variables directly influence both the onset and the rate of corrosion processes.

11 Summary of the appended Papers

Paper I: Spatial and time-dependent reliability analysis for post-tensioned concrete decks subjected to multiple failure modes

S. Celati, A. Natali, W. Salvatore, I. Björnsson, and S. Thöns, ‘Spatial and time-dependent reliability analysis for post-tensioned concrete decks subjected to multiple failure modes’, *Struct. Saf.*, vol. 117, p. 102634, Nov. 2025, doi: 10.1016/j.strusafe.2025.102634.

With the growing need to extend infrastructure lifespans under financial and environmental constraints, assessing the safety and remaining service life of ageing PT concrete bridge decks is becoming urgent. Existing approaches typically assess individual components, neglecting the spatial variability and system-wide interaction of degradation processes. A probabilistic approach is proposed to perform spatial and time-dependent system reliability analysis of PT decks. It explicitly models spatial correlation among sections and girders, temporal degradation of material properties, and multiple interacting failure modes. A key novelty of the paper is the incorporation of concrete quality into the corrosion model, distinguishing between “sound” and “bad” concrete conditions to reflect real-world variability. The activation phase of corrosion is modelled using coupled chloride diffusion and carbonation processes, while the propagation phase relies on experimentally calibrated models for pit-depth evolution. The approach is applied to a representative Italian PT bridge deck with four girders, resulting in an estimated service life of 39 years under sound concrete conditions, while poor-quality concrete reduces this to 20 years. Moreover, sensitivity analyses reveal that model uncertainties and corrosion activation time are the most influential parameters affecting reliability.

Paper II: Damage detection, localization, and quantification for steel cables of post-tensioned bridge decks

I. Mazzatura, W. Salvatore, S. Caprili, S. Celati, M. Mori, and M. Gammino, ‘Damage detection, localization, and quantification for steel cables of post-tensioned bridge decks’, *Structures*, vol. 57, p. 105314, Nov. 2023, doi: 10.1016/j.istruc.2023.105314.

An integrated inspection methodology is developed to evaluate the structural condition of PT concrete bridges, which constitute a large portion of Italy's

infrastructure network. These structures are prone to hidden defects, such as voids, poor grouting, and cable corrosion, that visual inspections cannot detect but that critically affect safety and performance. The study addresses the lack of standardised procedures for “Special Inspections” of PT systems in the Italian guidelines, offering a systematic approach based on non destructive test (NDT) techniques. The main novelty lies in defining and validating a complete in situ inspection framework covering: (i) cable layout identification, (ii) defect localisation and quantification, and (iii) estimation of the residual prestressing force. The methodology combines Ground Penetrating Radar (GPR), Ultrasonic Tomography (UT), X-ray imaging, and X-ray diffraction (XRD), supported by a hypergeometric-based sampling strategy to determine the number of tests. Laboratory experiments on reinforced concrete wall specimens, replicating different duct materials, defect types, and grouting conditions, were followed by applications to five real viaducts on the Italian highway network. The work shows that integrating multiple NDTs enables reliable diagnosis of hidden damage in PT bridges while reducing invasiveness and cost. The methodology provides a reproducible foundation for nationwide inspection protocols and enables future research to quantify the reliability of each technique and optimise their combined application.

Paper III: Statistical Data Analysis of Inspection Outcomes of Post-Tensioned Concrete Bridges in Italy

S. Celati, F. Mattei, W. Salvatore, and S. Thons, ‘Statistical Data Analysis of Inspection Outcomes of Post-Tensioned Concrete Bridges in Italy’, in *IABSE Congress Ghent 2025*, Ghent, Belgium, Aug. 2025, pp. 108–115. doi: 10.2749/ghent.2025.0108.

To characterise the actual defectiveness of existing PT girders, a large dataset of inspection campaigns carried out on PT bridge decks across Italy is analysed. In particular, the correlation and spatial distribution of defects (voids and corrosion spots) in PT bridge decks are computed, aiming to lay the groundwork for the development of probabilistic models for defect distribution in existing PT girders. The study analyses data from 53 bridges and 192 girders with simply supported configurations typical of Italian PT structures from the 1960s. Inspections followed the national multi-level methodology combining Ground Penetrating Radar (GPR), Ultrasonic Tomography, endoscopy, and, when necessary, destructive testing. A clear correlation between voids and corrosion is observed; however, corrosion can also occur in fully grouted ducts, indicating that injection completeness alone does not guarantee durability. The paper’s novel contribution lies in providing the first empirical, statistically grounded dataset on PT bridge defects in Italy. The results support the development of probabilistic spatial models for corrosion and void distribution, essential for reliability-based maintenance planning, optimised inspection strategies, and improved safety management of ageing PT bridge infrastructure.

Paper IV: Reliability Analysis of Post-Tensioned Concrete Bridges Using Inspection-Based Modelling of Grout Injection Voids

S. Celati, F. Mattei, W. Salvatore, and S. Thöns, ‘Reliability analysis of post-tensioned concrete bridges using inspection-based modelling of grout injection voids’, *Struct. Infrastruct. Eng.*, pp. 1–20, Apr. 2026, doi: 10.1080/15732479.2026.2651979.

Hidden deterioration in PT concrete bridges, where corrosion of prestressing steel may also be caused by poor grout injection, can lead to brittle, unexpected failures. Existing reliability assessments often rely on assumptions regarding defect location, lacking empirical validation from real-world data. To overcome these limitations, a data-driven probabilistic model for the number and spatial distribution of grout voids and corrosion spots in PT girders is developed. It derives from inspection results of 192 girders from 53 Italian bridges built in the 1960s. The model captures the joint rate and spatial correlation between voids and corrosion through copula-based modelling methods and is integrated into a novel spatial and time-dependent reliability approach. This approach enables simulation of corrosion initiation and propagation without the need for girder segmentation, treating defect positions and occurrences as random variables. The reliability analysis incorporates chloride diffusion and pitting corrosion models to represent degradation processes and employs Monte Carlo simulations to estimate failure probabilities over time. Using the proposed approach, a 30% longer service life is predicted compared with conventional analyses assuming a uniform defect distribution. The study’s novelty lies in integrating empirical inspection data into reliability analysis to produce a model for assessing PT bridge degradation. The approach supports data-informed maintenance planning, improved inspection prioritisation, and more accurate life-cycle safety evaluations for ageing bridge infrastructure.

Paper V: System Reliability Analysis of Reinforced Concrete Bridges Using Multiple Data Sources

Simone Celati, Agnese Natali, Walter Salvatore, Sebastian Thöns: ‘System Reliability Analysis of Reinforced Concrete Bridges Using Multiple Data Sources’, *Structural Safety* (Ready for submission)

The study responds to the increasing need to extend the service life of ageing infrastructure while maintaining safety and sustainability, presenting a comprehensive probabilistic approach for assessing and updating the reliability of RC bridges affected by corrosion-induced degradation. The proposed approach introduces two main novelties. First, it integrates multiple data sources, inspections, material testing, corrosion measurements, and traffic monitoring within a Bayesian updating framework, thereby reducing uncertainty in both degradation and load models. Second, it preserves the temporal characteristics of degradation processes during model updating and accounts for interactions among multiple failure modes and structural components through a nested system model. The methodology is

applied to a 60-year-old RC bridge in the Province of Pisa. A prior reliability analysis based on initial data indicates that the structure does not meet the target reliability index and would require immediate maintenance. Subsequently, the model is updated using new test data on concrete strength, carbonation, chloride diffusion, and weigh-in-motion traffic loads. Key results show that data-driven updating substantially improves reliability estimates, extending the bridge's technical service life by 18 years for major repair and 23 years for collapse prevention. The approach effectively captures the transition of dominant failure modes over time, from bending in sound conditions to shear and half-joint failures under advanced degradation, and shows how integrating inspection data, probabilistic modelling, and Bayesian updating can enhance decision-making for maintenance and life-cycle management, promoting both cost efficiency and structural safety in existing bridge networks.

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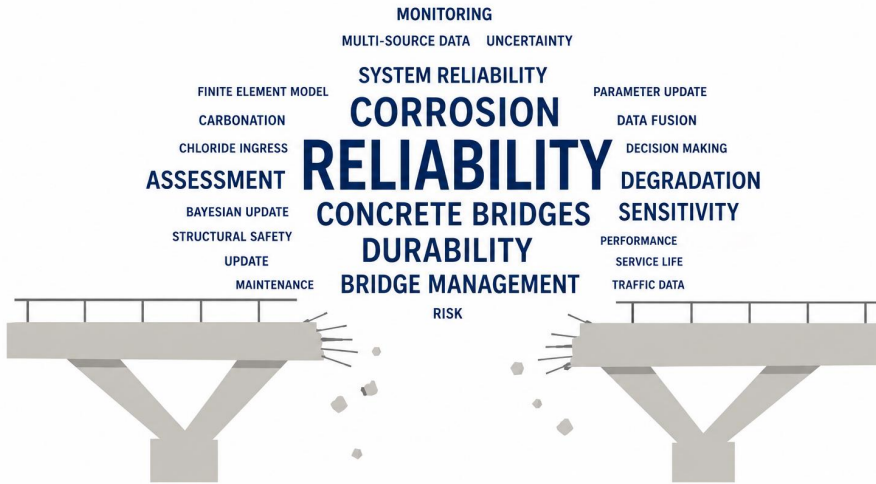
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