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DI PAVIA

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DICAr – Department of Civil Engineering and Architecture

**SEISMIC ASSESSMENT OF CORRODED BRIDGE  
STRUCTURES INCLUDING THE INTERACTION  
BETWEEN SEISMIC HAZARD AND DETERIORATION  
PHENOMENA**

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by

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*People's Dreams Don't Ever End!*





## ABSTRACT

Corrosion represents the most relevant deterioration phenomenon affecting Reinforced Concrete (RC) bridges all over the world. Given the complex, multivariate nature of the problem, the analysis of deteriorating RC bridges requires a multidisciplinary approach. Corrosion-induced deterioration is in fact associated with a number of effects, such as reinforcement section reduction, modification of steel and concrete mechanical properties, cover concrete cracking and spalling, increased rebar buckling effects, loss of bond strength and possible shift of failure mode. Seismic assessment of bridge structures is currently carried out according to the principles of Performance Based Earthquake Engineering (PBEE). The structural performance is expressed in terms of specific Decision Variables (DV). The PBEE framework represents a rational and rigorous tool for the seismic assessment of structures, allowing the analyst to explicitly take into account uncertainties in seismic hazard, structural response, fragility analysis and loss estimation. It is clear that the presence of deterioration phenomena leads to modification of structural properties and outputs of the framework. The integration of PBEE principles in lifetime seismic assessment is possible due to the modular nature of the former, allowing for including RC members deterioration in the process.

In this thesis, the phenomenon of corrosion is first introduced through a systematic literature review, summarizing the research efforts carried out in the past years and identifying research gaps. An application of state-of-the-art approaches on the seismic risk assessment of a deteriorating case study bridge presenting evidence of carbonation-induced corrosion is then proposed. The case study is analyzed both in pristine and corroded conditions in three ideal sites representative of low, moderate and high seismic hazard. Validation of the corrosion scenario is obtained through comparison with experimental data and seismic reliability is quantified through the PBEE framework. The complex nature of modeling corrosion effects on the cyclic response of RC columns is subsequently addressed through the development of an efficient and refined fiber-based modeling strategy, including critical factors such as reinforcement area reduction, decreased strength and ductility of steel and concrete, inelastic buckling, low-cycle fatigue and diminished bonding performance. Model validation is obtained through comparison with experimental data. Additional investigations include comparison with a less refined approach in both quasi-static and dynamic conditions. The issue of cover cracking on corrosion development is then discussed, specializing the problem to explicitly include the interaction between

environmental and seismic hazard in the analysis. The implications of concrete cracking are investigated both on durability and lifetime seismic performance assessment through probabilistic approaches. Given that the concept of Seismic Resilience represents the synthesis of modern PBEE, a specific framework tailored for deteriorating RC bridges is developed. Emphasis is placed on the effects of assumptions related to recovery phase uncertainty and time-varying initial functionality. The applicability of the approach is demonstrated through application to new and existing bridge configurations. Finally, an application of the principles of Direct Displacement Based Assessment (DDBA) to lifetime seismic assessment is proposed, aiming at reducing the computational burden of the analysis. State-of-the-art approaches for simplified seismic analysis are collected and integrated into the PBEE framework. Different structural configurations and exposure conditions are analyzed, and the results are compared to those obtained through Non-Linear Time History Analysis (NLTHA). An application to uncertainty analysis is then presented and the results are discussed in terms of fragility curves, seismic risk, and Expected Annual Loss Ratio (EALR).



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## LIST OF SYMBOLS AND ACRONYMS

|             |                                                                    |
|-------------|--------------------------------------------------------------------|
| $\emptyset$ | = Bar diameter                                                     |
| 1DME        | = Uniaxial Inertial Macroelements                                  |
| $2N_f$      | = Number of half cycles to failure                                 |
| $A$         | = Cross-section area of rubber                                     |
| $A(t)$      | = Aging function                                                   |
| $a(t)$      | = Pit width                                                        |
| $A_0$       | = Steel area in pristine condition                                 |
| $A_{C,P}$   | = Residual steel area in a given pit location                      |
| $A_{C,U}$   | = Residual steel area computed through the uniform corrosion model |
| $A_{CR}$    | = Residual steel area after concrete cracking                      |
| ADRS        | = Acceleration-Displacement Response Spectrum                      |
| $A_{R,P}$   | = Residual steel area in presence of pitting                       |
| $AvgSd_k$   | = Average Spectral Displacement                                    |
| $b_0$       | = Section width in pristine condition                              |
| $b_c$       | = Parameter of $k_c$                                               |
| $b_f$       | = Increased section width due to corrosion                         |
| $c$         | = Cover concrete depth                                             |
| $C(x,t)$    | = Chloride Content at depth $x$ and time $t$                       |
| $c'$        | = Soil cohesion                                                    |
| $C_0$       | = Initial Chloride Content                                         |
| $C_{crit}$  | = Critical chloride content threshold                              |
| CDI         | = Corrosion Damage Index                                           |
| CDR         | = Compound Damage Ratio                                            |



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|                  |                                                                                        |
|------------------|----------------------------------------------------------------------------------------|
| $C_f$            | = Coefficient equal to 1 or 2 for single or double bending piers                       |
| CFRP             | = Carbon Fiber Reinforced Polymer                                                      |
| CIDS             | = Corrosion-Induced Damage State                                                       |
| $C_{rep}$        | = Replacement Cost                                                                     |
| $C_s$            | = CO <sub>2</sub> concentration                                                        |
| CS               | = Conditional Spectrum                                                                 |
| $C_{s,atm}$      | = Atmospheric CO <sub>2</sub> concentration                                            |
| $C_{s,emi}$      | = Additional CO <sub>2</sub> concentration due to emission sources                     |
| $C_{s,\Delta x}$ | = Chloride Content at depth $\Delta x$                                                 |
| CV               | = Coefficient of Variation                                                             |
| $c_{W,I}$        | = Corrosion Damage Factor for the i-th CIDS                                            |
| $D$              | = Bar diameter                                                                         |
| D/C              | = Demand to Capacity ratio                                                             |
| $D_0$            | = Pristine bar diameter                                                                |
| $d_{60}$         | = Displacement associated with a pressure of 60 MPa on the bearings                    |
| $D_{app}(t)$     | = Apparent Chloride Diffusion Coefficient at time $t$                                  |
| $D_{av}$         | = Average chloride diffusion coefficient in cracked concrete                           |
| $d_{bl}$         | = Minimum diameter of longitudinal bars                                                |
| DBPP             | = Displacement Based Pseudo Pushover                                                   |
| $D_{cr}$         | = Chloride diffusivity of a crack                                                      |
| $d_d$            | = Displacement corresponding to significant residual displacements of foundation piles |
| DDBA             | = Direct Displacement Based Assessment                                                 |
| DDBD             | = Direct Displacement Based Design                                                     |
| $D_{eq}$         | = Equivalent Diffusion Coefficient                                                     |

---

|                    |                                                                                             |
|--------------------|---------------------------------------------------------------------------------------------|
| $d_{fr}$           | = Displacement corresponding to the onset of relative sliding between neoprene pad and deck |
| $d_{gap}$          | = Displacement corresponding to gap closure                                                 |
| $D_{rm,0}(t_0)$    | = Chloride Diffusion Coefficient at reference time $t_0$                                    |
| $d_{res}$          | = Residual drift                                                                            |
| $d_{rs,i}(t)$      | = Depth of corrosion attack on the i-th bar                                                 |
| DS                 | = Damage State                                                                              |
| $DS_p$             | = Probable Damage State                                                                     |
| $d_u$              | = Ultimate displacement                                                                     |
| $d_{u,ab}$         | = Ultimate backwall displacement                                                            |
| $d_{uns}$          | = Displacement corresponding to deck unseating                                              |
| DV                 | = Decision Variable                                                                         |
| $d_y$              | = Yielding displacement                                                                     |
| $d\lambda(IM > x)$ | = Derivative of the site seismic hazard curve                                               |
| EAL                | = Expected Annual Losses                                                                    |
| EALR               | = Expected Annual Loss Ratio                                                                |
| EDP                | = Engineering Demand Parameter                                                              |
| $erf$              | = Gaussian Error Function                                                                   |
| $E_s$              | = Steel Young modulus                                                                       |
| F                  | = Force                                                                                     |
| $F_{bw}$           | = Backwall strength                                                                         |
| $f_c$              | = Concrete compressive strength                                                             |
| $f_{c,corr}$       | = Reduced cover concrete compressive strength                                               |
| $f_c^{t_c}$        | = Confined concrete compressive strength in corroded condition                              |
| $f_{ctm}$          | = Mean concrete tensile strength                                                            |
| $f_e$              | = Parameter of $W(t)$                                                                       |

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|            |                                                  |
|------------|--------------------------------------------------|
| $F_{fr}$   | = Friction force                                 |
| $f_l^c$    | = Effective lateral confining pressure           |
| $f_u$      | = Steel ultimate strength                        |
| $f_y$      | = Steel yield strength                           |
| $f_{yb}$   | = Transverse reinforcement yield strength        |
| $G$        | = Rubber shear modulus                           |
| $g(X,t)$   | = Limit State Function                           |
| $g_e$      | = Parameter of $W(t)$                            |
| GM         | = Ground Motion                                  |
| GMPE       | = Ground Motion Prediction Equation              |
| $H(.)$     | = Heaviside step function                        |
| $h_{bw}$   | = Backwall height                                |
| $i_{corr}$ | = Corrosion Current Density                      |
| IM         | = Intensity Measure                              |
| $K$        | = Shear stiffness of bearing devices             |
| $k_1$      | = Coefficient accounting for bond properties     |
| $k_2$      | = Coefficient accounting for strain distribution |
| $k_{bw}$   | = Backwall stiffness                             |
| $k_c$      | = Execution Transfer Variable                    |
| $k_{carb}$ | = Carbonation penetration rate coefficient       |
| $k_e$      | = Environmental Transfer Variable                |
| $K_{el}$   | = Elastic stiffness                              |
| $k_{in}$   | = Initial Stiffness                              |
| $k_{sec}$  | = Secant stiffness                               |
| $k_t$      | = Transfer Variable                              |

---

|               |                                                                        |
|---------------|------------------------------------------------------------------------|
| $k_{ld}$      | = Factor accounting for load duration                                  |
| $L$           | = Loss                                                                 |
| LCC           | = Life Cycle Cost                                                      |
| $L_D$         | = Direct Losses                                                        |
| $L_{eff}$     | = Bar Buckling Length                                                  |
| LHS           | = Latin Hypercube Sampling                                             |
| $L_p$         | = Plastic Hinge length                                                 |
| $L_{pil}$     | = Length of foundation piles                                           |
| LSF           | = Least-Squares Fitting                                                |
| $L_{sp}$      | = Strain Penetration Length                                            |
| M             | = Magnitude                                                            |
| $M^*$         | = Modal Participating Mass                                             |
| MAFE          | = Mean Annual Frequency of Exceedance                                  |
| $m_{eff}$     | = Effective Mass                                                       |
| MLE           | = Maximum Likelihood Estimation                                        |
| MRP           | = Mean Return Period                                                   |
| MSA           | = Multiple Stripe Analysis                                             |
| $n$           | = Buckling mode                                                        |
| $n_{bars}$    | = Number of bars on the section side                                   |
| $n_{carb}$    | = Carbonation rate parameter                                           |
| NLTHA         | = Nonlinear Time History Analysis                                      |
| $n_{pil}$     | = Number of foundation piles parallel to bridge longitudinal axis      |
| $n_{pt}$      | = Number of foundation piles perpendicular to bridge longitudinal axis |
| $P(F IM = x)$ | = Fragility Function                                                   |
| $p(t)$        | = Pit depth                                                            |

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|                  |                                                                |
|------------------|----------------------------------------------------------------|
| $P[ ]$           | = Probability                                                  |
| $P_{100}$        | = Probability of exceeding a given CIDS for $t=100$ years      |
| PBEE             | = Performance Based Earthquake Engineering                     |
| PC               | = Prestressed Concrete                                         |
| $p_{crit}$       | = Critical pit depth                                           |
| PL               | = Performance Level                                            |
| PSHA             | = Probabilistic Seismic Hazard Analysis                        |
| $Q(t)$           | = Functionality Indicator                                      |
| $Q_0$            | = Initial Functionality                                        |
| $Q_r$            | = Residual Functionality                                       |
| $Q_t$            | = Target Functionality                                         |
| R                | = Resilience Index                                             |
| $R^{-1}_{ACC,0}$ | = Inverse Effective Carbonation Resistance of Concrete         |
| RC               | = Reinforced Concrete                                          |
| RH               | = Relative Humidity                                            |
| RV               | = Random Variable                                              |
| $s$              | = Transverse reinforcement spacing                             |
| $S_a$            | = Spectral Acceleration                                        |
| $S_a(T_1)$       | = Spectral acceleration associated with the fundamental period |
| $s_{carb}$       | = Thickness of the carbonated layer                            |
| SD               | = Severe Damage                                                |
| $S_d$            | = Spectral Displacement                                        |
| SF               | = Structural Failure                                           |
| $s_{r,max}$      | = Maximum crack spacing                                        |
| SRA              | = Site Response Analysis                                       |

---

|            |                                                          |
|------------|----------------------------------------------------------|
| SSI        | = Soil-Structure Interaction                             |
| $S_u$      | = Soil undrained shear strength                          |
| $s_y$      | = Slip at yield stress                                   |
| T          | = Period                                                 |
| $t$        | = Time                                                   |
| $t_0$      | = Reference Time                                         |
| $T_c$      | = Conditioning Period                                    |
| $t_c$      | = Period of curing                                       |
| $t_{cr}$   | = Time to corrosion-induced cracking                     |
| $T_d$      | = Equivalent Downtime                                    |
| $T_{eff}$  | = Effective period                                       |
| $t_b$      | = Control Time                                           |
| $t_i$      | = Corrosion Initiation Time                              |
| $t_p$      | = Propagation Period                                     |
| $T_r$      | = Return Period                                          |
| $t_r$      | = Total thickness of rubber layers                       |
| $t_{rec}$  | = Recovery time                                          |
| UPD        | = Usability Preventing Damage                            |
| $V_b$      | = Base Shear                                             |
| $V_{s,30}$ | = Shear wave velocity in the first 30 m of soil          |
| $V_{s,m}$  | = Average value of soil small-strain shear wave velocity |
| $V_u$      | = Shear Resistance                                       |
| $w(t)$     | = Crack width                                            |
| $W(t)$     | = Weather Function                                       |
| $w/b$      | = Water to binder ratio                                  |

**Seismic Assessment of Corroded Bridge Structures Including the Interaction Between Seismic xxxv  
Hazard and Deterioration Phenomena**

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|                |                                                                           |
|----------------|---------------------------------------------------------------------------|
| $w/c$          | = Water to cement ratio                                                   |
| $w_m$          | = Mean crack width                                                        |
| $w_{sev}$      | = Critical crack width for severe cracking                                |
| $w_{spa}$      | = Critical crack width for spalling                                       |
| $x_0$          | = Amount of corrosion that triggers cracking in case of uniform corrosion |
| $x_c(t)$       | = Carbonation front depth at time $t$                                     |
| $z_b$          | = Depth of lower boundary of soil layers                                  |
| $\alpha$       | = Material constant of Coffin-Manson model                                |
| $a_e$          | = Ratio between steel elastic modulus and concrete secant elastic modulus |
| $a_{fu}$       | = Empirical coefficient                                                   |
| $a_{fy}$       | = Empirical coefficient                                                   |
| $\alpha_s$     | = Parameter of the bond-slip model                                        |
| $a_c$          | = Empirical coefficient                                                   |
| $\beta_c$      | = Model-related dispersion component due to construction quality          |
| $\beta_{comp}$ | = Dispersion of component fragility curves                                |
| $\beta_{gm}$   | = Dispersion due to record-to-record variability                          |
| $\beta_i$      | = Dispersion of a fragility function                                      |
| $\beta_{ls}$   | = Dispersion due to uncertainty in DS definition                          |
| $\beta_{mod}$  | = Dispersion due to modeling uncertainty                                  |
| $\beta_q$      | = Model-related dispersion component due to model quality                 |
| $\Gamma$       | = Modal Participation Factor                                              |
| $\gamma$       | = Soil unit weight                                                        |
| $\Delta_{eff}$ | = Effective Displacement                                                  |
| $\Delta_{eq}$  | = Equivalent displacement                                                 |
| $\Delta_i$     | = Idle time                                                               |

---

|                      |                                                           |
|----------------------|-----------------------------------------------------------|
| $\Delta_i'$          | = Displacement Profile                                    |
| $\Delta Q$           | = Functionality Loss                                      |
| $\delta_s$           | = Damage function                                         |
| $\Delta_s$           | = Difference between pristine and residual bar section    |
| $\Delta_t$           | = Target displacement                                     |
| $\Delta t$           | = Time difference                                         |
| $\Delta_u$           | = Pier ultimate displacement                              |
| $\Delta x$           | = Depth of the convection zone                            |
| $\Delta_y$           | = Pier yielding displacement                              |
| $\varepsilon$        | = Epsilon parameter                                       |
| $\varepsilon_t$      | = Average tensile strain in cracked concrete              |
| $\varepsilon_{t0}$   | = Strain at peak concrete compressive strength            |
| $\varepsilon_{su}^c$ | = Transverse reinforcement ultimate strain                |
| $\varepsilon_{cu}^c$ | = Confined concrete ultimate strain in corroded condition |
| $\varepsilon_p$      | = Plastic strain amplitude                                |
| $\varepsilon_{su}$   | = Steel ultimate strain                                   |
| $\varepsilon_t$      | = Error term                                              |
| $\varepsilon_y$      | = Steel yielding strain                                   |
| $\eta$               | = Spectral reduction factor                               |
| $\theta$             | = Material constant of Coffin-Manson model                |
| $\theta_{DS_p}$      | = Median of the fragility curve corresponding to $DS_p$   |
| $\theta_i$           | = Median value of a fragility function                    |
| $\theta_y$           | = Chord rotation corresponding to pier yielding           |
| $\lambda$            | = Slenderness Ratio                                       |
| $\lambda_f$          | = Failure Rate                                            |



|                |                                             |
|----------------|---------------------------------------------|
| $\mu$          | = Friction coefficient                      |
| $\mu_{D,j}$    | = Damage ratio for a given DS               |
| $\mu_k$        | = Ductility demand                          |
| $\mu_{pe}$     | = Post-event available ductility            |
| $\xi$          | = Equivalent Viscous Damping                |
| $\xi_{eff}$    | = Effective damping                         |
| $\rho_{p,eff}$ | = Effective Tension Area                    |
| $\rho_s$       | = Transverse reinforcement volumetric ratio |
| $\sigma_s$     | = Stress in Tension reinforcement           |
| $\sigma_v$     | = Vertical compressive stress in [MPa]      |
| $\sigma_y$     | = Steel yield strength in [MPa]             |
| $\tau$         | = Normalized time                           |
| $\nu_{rs}$     | = Ratio of volumetric expansion of oxides   |
| $\varphi(.)$   | = Standard normal cumulative distribution   |
| $\varphi'$     | = Soil angle of shearing resistance         |
| $\Phi_c$       | = Modal Displacement of the control node    |
| $\Phi_i$       | = Mode Shape                                |
| $\varphi_u$    | = Ultimate curvature                        |
| $\varphi_y$    | = Yield curvature                           |
| $\psi$         | = Bar Section Loss                          |



# 1.INTRODUCTION

## 1.1 MOTIVATION

Corrosion represents the most relevant deterioration phenomenon affecting Reinforced Concrete (RC) members, reducing durability and safety over time. Corrosion of embedded steel reinforcement may be induced by chloride ingress or concrete carbonation, depending on the exposure conditions. The effects of corrosion on RC members include a number of different aspects, summarized in Figure 1.1, with the most obvious one being the reduction of steel reinforcement area. The mechanical properties of corroded steel generally decrease as corrosion severity increases. The expansion of corrosion products may lead to cracking and, in some cases, spalling of concrete due to internal pressure, reduction of bond strength between steel and decrease of damaged concrete compressive strength. Finally, corrosion of transverse reinforcement is associated with lower confinement level, ultimately resulting in modification of confined concrete ultimate strength and strain.

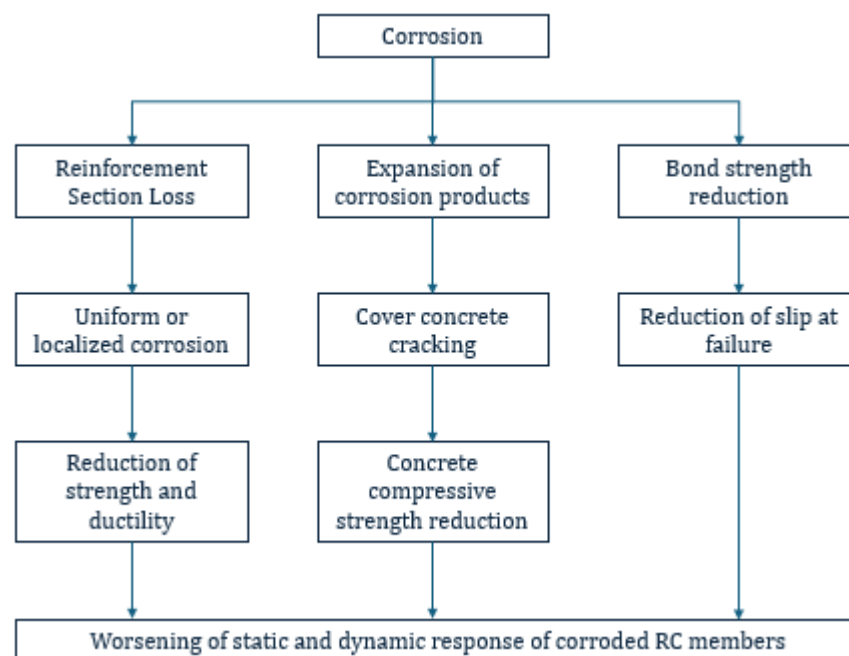


Figure 1.1. Effects of corrosion on RC elements deterioration.

It is worth noting that RC bridges are normally exposed to harsh environmental conditions and, as most of the existing RC bridges are approaching or have in some cases exceeded their intended useful lifetime, assessing their structural performance accounting for deterioration represents an urgent task [1]. The collapse of the Polcevera Viaduct on August 14, 2018, resulting in 43 fatalities, is a notable example of the catastrophic consequences of deterioration on RC bridges. The viaduct, which was a critical component of the Italian road network, exhibited significant deterioration with widespread corrosion [2].

The economic side also plays a crucial role while discussing the importance of corrosion. As revealed by a study for the Federal Highway Administration, the combined direct and indirect cost of corrosion was over \$550 billion [3]. Concerning the management of road networks, existing aging RC bridges should be maintained in cost-effective way and, at the same time, new structures should be designed following durability and sustainability principles [4]. The challenges posed by deteriorating RC structures to structural engineers all over the world require a complex and multidisciplinary approach, both in research and practice, combining materials science, structural engineering and predictive modeling, in order to effectively address the problem.

Bridge structures reported significant damage during past seismic events. One of the most famous cases is represented by the Cypress Street Viaduct in Oakland, California, which collapsed during the 1989 Loma Prieta earthquake causing 42 fatalities [5], but other remarkable examples can be found referring to the 1971 San Fernando earthquake [6]. Given that bridges represent crucial nodes of road networks, preventing their collapse is fundamental but it is not the only goal that should be included in the design. In fact, traffic disruption due to excessive damage on bridge structures leads to significant economic losses and, on the societal side, may slow down rescue operations. The first steps in this direction have been undertaken with the Vision 2000 report [7], where the concept of different Performance Levels (PLs) for different seismic hazard levels was introduced. In this sense, the document represents the first introduction of Performance Based Earthquake Engineering (PBEE). Modern PBEE, developed starting from the research of Cornell and Krawinkler [8], expresses the seismic performance of a structure through Decision Variables (DVs) reflecting the system performance. The DVs are usually expressed in economic terms, such as Expected Annual Losses (EAL), but may also include the Mean Annual Frequency of Exceedance (MAFE) of a given PL. The modern PBEE framework represents a rational and rigorous tool for the seismic assessment of structures, allowing the analyst to explicitly take into account uncertainties in seismic hazard, structural response, fragility analysis and loss estimation. Four sequential modules can be identified in the PBEE framework:

- Seismic Hazard: described through Probabilistic Seismic Hazard Analysis (PSHA);
- Structural Analysis: estimation of seismic demand conditional to hazard intensity level;

- Fragility Analysis: description of the damage probability of components/system;
- Loss Analysis: estimation of specific DVs.

While PBEE represents an elegant way to perform seismic assessment of RC bridges, it is clear that the presence of deterioration phenomena leads to modification of structural properties and, most likely, of the outputs of the framework. It is worth noting that, in the context of lifetime seismic assessment of RC bridges [9], PBEE represents a valuable and theoretically consistent tool. The integration of PBEE principles in lifetime seismic assessment is possible due to the modular nature of the former, allowing for implementing RC members deterioration in Structural and Fragility Analysis modules, representing the basis for subsequent steps.

It is however worth noting that, despite the significant research effort, a number of challenges still exist in the field of lifetime seismic assessment of RC bridges, briefly summarized in the following. Robust estimation of parameters involved in corrosion initiation and propagation is needed, in order to increase confidence in the subsequent analysis results. Reliable deterioration laws for steel and concrete are needed, as well as consistent modeling approaches for corroded members. PBEE-based lifetime seismic performance assessment should be rationally integrated in order to effectively communicate the relevance of deterioration of RC structures to stakeholders and owners. Simplified approaches should be developed in order to reduce the inherent computational effort of lifetime seismic assessment.

## **1.2 SCOPE AND OBJECTIVES**

Based on the previous discussion, the primary objectives of this thesis can be efficiently summarized as:

- Investigating the implications of corrosion-induced deterioration on the seismic performance of RC bridges through the application of state-of-the-art approaches on a real case study, validating the results through comparison with experimental data collected in site.
- Developing, and validating through experimental data, a practical and refined numerical modeling strategy for the seismic simulation of corroded RC bridge piers, including reinforcement section reduction, material properties modification, bond-slip effects, low-cycle fatigue and inelastic buckling. Comparing the results with those obtained through less refined approaches in order to understand the relative importance of key modeling assumptions.
- Incorporating concrete cracking in the chloride-induced corrosion evolutionary model and applying it for investigating the mutual interaction between environmental and seismic hazard. Including the effects of possible cracking due to serviceability level

earthquakes in corrosion simulations and understanding the implications on durability and PBEE-based lifetime seismic assessment.

- Integrating deterioration in the PBEE framework and developing a Seismic Resilience assessment framework tailored for deteriorating RC bridges, aiming at facilitating the communication of results of lifetime seismic performance assessment to stakeholders and owners. Investigating the impact of uncertainty in the recovery process on the Seismic Resilience metrics through a simulation-based approach. Applying the framework to a set of modern and existing bridge configurations, in different exposure scenarios.
- Proposing and evaluating lifetime seismic assessment-oriented extensions of simplified assessment methodologies based on equivalent description of the bridge system. Carrying out validation through comparison with NLTHA-based approaches and discussing the possible application of simplified approaches for large-scale assessment and preliminary screening of bridge structures, including deterioration effects.

The research objectives introduced above are oriented towards improving some critical aspects of life cycle engineering, following a rigorous and consistent framework. Each research question represents an independent contribution to the field, exploiting the modular nature of lifetime seismic performance assessment and allowing for independent evaluation and implementation of the research.

### **1.3 ORIGINAL CONTRIBUTIONS**

Based on the previously introduced research objectives, the original contributions of the thesis can be summarized as:

- A case study application of state-of-the-art structural and corrosion damage modeling approaches, demonstrating the effects of corrosion-induced deterioration on seismic risk assessment and strengthened by experimental validation based on in-site testing.
- A refined numerical modeling strategy for the seismic simulation of corroded RC bridge piers, including deterioration effects on critical facets of the cyclic response such as inelastic buckling, low-cycle fatigue and bond-slip, validated through the simulation of experimental quasi-static cyclic tests on corroded and uncorroded specimens.
- A methodology for incorporating concrete cracking induced by serviceability level earthquakes on corrosion evolution simulation, including quantification of the relevance of this aspect on durability and lifetime seismic performance assessment.
- A modern and PBEE-consistent Seismic Resilience framework, incorporating state-of-the-art approaches in each of its submodules and explicitly including corrosion-induced deterioration effects, allowing for transparent and clear communication between analysts and stakeholders.

- A simplified lifetime seismic assessment methodology, incorporating the most recent research of the DDBA field, suitable for the rapid assessment of common bridge configurations and for first-level screening of large bridge portfolios.

#### **1.4 THESIS OUTLINE**

The thesis includes nine chapters, addressing the research questions previously introduced. Each of the core chapters (Chapters 3-8) is organized as an independent research work addressing one of the key topics previously introduced. For this reason, each chapter includes a brief literature review of the corresponding topic, and the relevant models are recalled aiding the clarity of the research. For the sake of convenience, the organization of the thesis and the connections between chapters are depicted in Figure 1.2. The present introduction (Chapter 1) addressed research motivation, scope and objectives of the thesis and describes the structure of the dissertation.

Chapter 2 is a systematic literature review, summarizing the research efforts carried out in the past years concerning corrosion characterization and its effects on RC bridges. Corrosion evolutionary models for chloride- and carbonation-induced corrosion are recalled and discussed, highlighting the main critical points. Corrosion-induced deterioration of RC members is then presented, discussing available deterioration laws and corrosion morphologies. A review of published research on lifetime seismic assessment of RC bridges is then proposed. Finally, based on the literature survey, research gaps needing further research are identified.

Chapter 3 presents the seismic risk assessment of a case study bridge, analyzed in three ideal sites representative of low, moderate and high seismic hazard and presenting evidence of carbonation-induced corrosion. The case study bridge is analyzed both in pristine and corroded conditions, in order to highlight the impact of deterioration on the seismic response. The results of an extensive experimental campaign are used to validate the corrosion scenario. State-of-the-art numerical models, including refined description of bearings response and Soil-Structure Interaction are developed and seismic reliability is quantified through the PBEE framework. The results reported in this chapter are part of the collaborative research project DPC-ReLUIIS 2022-24 WP3 on seismic risk and reliability of existing bridges.

In Chapter 4, the complex multivariate nature of modeling corrosion effects on the cyclic response of RC columns is addressed. An efficient and refined fiber-based modeling strategy specifically designed for the seismic analysis of corroded RC piers and including critical factors such as reinforcement area reduction, decreased strength and ductility of steel and concrete, inelastic buckling, low-cycle fatigue and diminished bonding performance is developed in this chapter. Model validation is achieved through comparison with quasi-static cyclic tests on both uncorroded and corroded specimens. The

performance of the modeling strategy is compared with that of a less refined approach, highlighting the differences in the simulation of the hysteretic response. Finally, the influence of corrosion on the seismic response of RC piers is investigated through Nonlinear Time History Analysis (NLTHA).

The research carried out in Chapter 5 addresses the issue of cover concrete cracking influence on corrosion development. Focusing the issue on lifetime seismic assessment, the interaction between environmental and seismic hazard is traditionally studied only as the evolution of DVs over time on deteriorating structures. However, serviceability level earthquakes may induce cover concrete cracking on bridge piers, possibly affecting corrosion initiation and structural integrity if not promptly repaired. The chapter details a practical methodology to investigate the mutual interaction between environmental and seismic hazard. Concrete cracking is taken into account through the inclusion of an equivalent diffusion coefficient in the chloride diffusion model. The implications of concrete cracking are investigated both on durability and lifetime seismic performance assessment. For the former, a probabilistic approach is used to predict corrosion evolution and corrosion-induced damage. For the latter, PBEE-based lifetime seismic assessment is performed. The results are carefully scrutinized and critically discussed, quantifying the differences with respect to the benchmark approach, assuming uncracked concrete.

Chapter 6 introduces a Seismic Resilience assessment framework for deteriorating RC bridge structures. Seismic Resilience represents the synthesis of modern PBEE, being naturally connected to relevant facets of disaster risk mitigation, such as economic losses, network downtime, rescue delay and restoration of normal functionality after the event and allows for transparent and clear communication between stakeholders and analysts, effectively highlighting the role played by deterioration on the seismic performance of a bridge. Considering structures exposed to harsh environmental conditions, the structural performance is not constant over time, affecting initial functionality, recovery phase and seismic resilience. The chapter defines a practical and PBEE-consistent Seismic Resilience assessment framework, discussing its main theoretical concepts. The effects of the assumptions related to recovery time uncertainty and initial functionality are thoroughly investigated through the application for the lifetime seismic assessment of a prototype bridge configuration.

Chapter 7 presents the application of the Seismic Resilience assessment framework to existing Italian bridge configurations. Four structural configurations are derived from a real case study bridge, in order to study both regular and irregular schemes. Three progressively more severe environmental exposure scenarios are identified, and lifetime seismic analysis is carried out for each configuration. The chapter aims at investigating the influence of exposure conditions on the long-term seismic resilience assessment of bridge structures and at studying different structural configurations representative of existing Italian bridges.



In Chapter 8, the principles of Direct Displacement Based Assessment (DDBA) are applied in a lifetime seismic assessment framework, with the goal of reducing the computational cost associated with lifetime seismic assessment of bridge structures. State-of-the-art approaches for the simplified description of the seismic response of bridge structures are collected and applied to obtain the Engineering Demand Parameter (EDP) distributions, then integrated into the PBEE framework. Validation is carried out through comparison with NLTHA-based results. Different structural configurations and exposure conditions are considered. An application to uncertainty analysis in the lifetime assessment is carried out, and the results are discussed in terms of fragility curves, seismic risk, and Expected Annual Loss Ratio (EALR).

Finally, Chapter 9 summarizes the results and conclusions of this dissertation and presents future developments of the existing work.

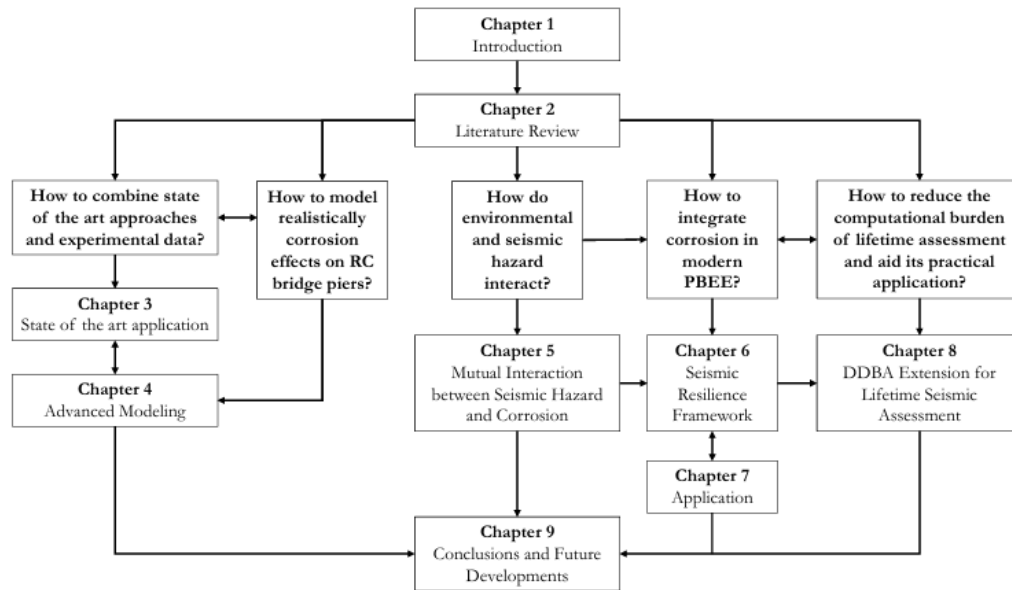


Figure 1.2. Organization of the thesis.

## 1.5 CASE STUDIES

To further clarify the structure of the dissertation, this paragraph summarizes the main structural characteristics of the case studies and the analysis performed on them through the core chapters and the appendixes. The dissertation includes a total of seven case study bridges. Table 1.1 reports the main characteristics of the case studies and the chapters where they have been analyzed, while Table 1.2 summarizes the analysis performed on each

case study. It is worth pointing out the Case Studies 1, 3 and 7 represent real bridge configurations, with information retrieved from original design documents. Case Study 2 is a prototype bridge designed according to current standards. Finally, Case Studies 4, 5 and 6 have been defined based on Case Study 3, varying pier height and deck skewness in order to study the long-term performance of different structural configurations.

**Table 1.1. Main characteristics of the case studies.**

| <i>Bridge ID</i> | <i>Numbers of spans</i> | <i>Length [m]</i> | <i>Deck</i>       | <i>Pier Configuration</i> | <i>Chapters</i> |
|------------------|-------------------------|-------------------|-------------------|---------------------------|-----------------|
| 1                | 3                       | 43.8              | PC girders        | Four-column bent          | 3               |
| 2                | 2                       | 60                | PC girders        | Three-column bent         | 5, 6, 8, App. C |
| 3                | 4                       | 120               | PC girders (skew) | Wall                      | 7               |
| 4                | 4                       | 120               | PC girders        | Wall                      | 7               |
| 5                | 4                       | 120               | PC girders (skew) | Wall                      | 7               |
| 6                | 4                       | 120               | PC girders        | Wall                      | 7, 8            |
| 7                | 4                       | 50                | RC girders        | Two-column bent           | App. A          |

**Table 1.2. Analysis performed on the case studies. Performing deterioration analysis implies that all of the analyses have been performed with and without deterioration.**

| <i>Bridge ID</i> | <i>Deterioration Analysis</i> | <i>Fragility Analysis</i> | <i>Risk Analysis</i> | <i>Loss Analysis</i> | <i>Resilience Analysis</i> | <i>DDBA</i> |
|------------------|-------------------------------|---------------------------|----------------------|----------------------|----------------------------|-------------|
| 1                | X                             | X                         | X                    |                      |                            |             |
| 2                | X                             | X                         | X                    | X                    | X                          | X           |
| 3                | X                             | X                         | X                    | X                    | X                          |             |
| 4                | X                             | X                         | X                    | X                    | X                          |             |
| 5                | X                             | X                         | X                    | X                    | X                          |             |
| 6                | X                             | X                         | X                    | X                    | X                          | X           |
| 7                |                               | X                         | X                    |                      |                            |             |

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## **2. CORROSION-INDUCED DETERIORATION AND TIME-DEPENDENT ASSESSMENT OF STRUCTURES – STATE OF THE ART**

### **2.1 INTRODUCTION**

Corrosion represents the main deterioration phenomenon in Reinforced Concrete (RC) elements, affecting durability and safety of structures over time. Depending on the environmental conditions, corrosion can be induced by carbonation or chloride ingress, each impacting the longevity of RC structures in distinct ways. The primary effects of corrosion include the deterioration of reinforcing bars embedded in concrete, cracking and eventual spalling of concrete due to the expansive pressure of corrosion products, and a reduction in the bond strength between steel and concrete. Remarkably, as most of the existing structures are approaching or have already exceeded their useful lifetime, assessing the capacity of corroded RC elements represents an impelling task [1].

The catastrophic consequences of deterioration and corrosion on RC elements have been observed in bridge collapses of the past few years. A notable example is the Polcevera Viaduct, a critical infrastructure in the Italian road network, which collapsed on August 14, 2018, resulting in 43 fatalities. The bridge exhibited significant deterioration with widespread corrosion [2].

Challenges associated with corrosion are related to different aspects. On one hand, it is necessary to maintain existing aging RC structures in a cost-effective way. On the other hand, new structures have to be designed in durable and sustainable way [3]. Addressing the challenges posed by corrosion and deterioration of RC structures is complex and requires a multidisciplinary approach, combining materials science, civil engineering and predictive modeling. The benefits associated with advances in corrosion studies include reduced maintenance cost, optimized maintenance plans and increased service life of RC structures.

The following sections summarize the state of the art of the field. Corrosion in RC elements is presented, highlighting the differences between chloride- and carbonation-induced corrosion, its evolution over time and the main influencing factors. Deterioration induced by corrosion in RC elements is then detailed, covering all of the main corrosion-induced

damage studied in the literature. The main aspects of time-dependent assessment are presented. The goal of this chapter is to present the state of the art in a critical sense and identify areas needing further research.

## 2.2 CORROSION IN RC ELEMENTS

### 2.2.1 Phases of the Corrosion Process

Corrosion is conventionally divided into two phases, corrosion propagation and corrosion initiation [4]. The process is different considering chloride- and carbonation-induced corrosion. Considering chloride-induced corrosion, Figure 2.1 represents the service life of a corroding RC element. During the initiation phase, chloride ions penetrate the concrete cover and, once that a critical threshold is reached at the depth of the reinforcement layer, initiate the corrosion process. A number of parameters, such as concrete cover depth, surface chloride concentration, quality of the concrete cover, type of reinforcing steel and chloride threshold level, define corrosion initiation time [5]. During the propagation phase, corrosion products expand and the stresses that originate lead to cracking, delamination and spalling of the concrete cover. Other effects induced by corrosion are loss of ultimate strength, loss of bond between steel and concrete and member failure [5]. Considering carbonation-induced corrosion, the carbonation process starts on the external surface of exposed members and penetrates into the concrete volume originating a low pH front. Corrosion starts once that the carbonated front reaches the depth of the reinforcement layer. The factors influencing the carbonation rate include humidity,  $\text{CO}_2$  concentration, temperature, concrete composition and thickness of the concrete cover [6]. Once the carbonation depth reaches the cover concrete thickness, corrosion starts if a sufficient quantity of water and oxygen is present. The following paragraphs address in detail the various aspects regarding chloride- and carbonation- induced corrosion.

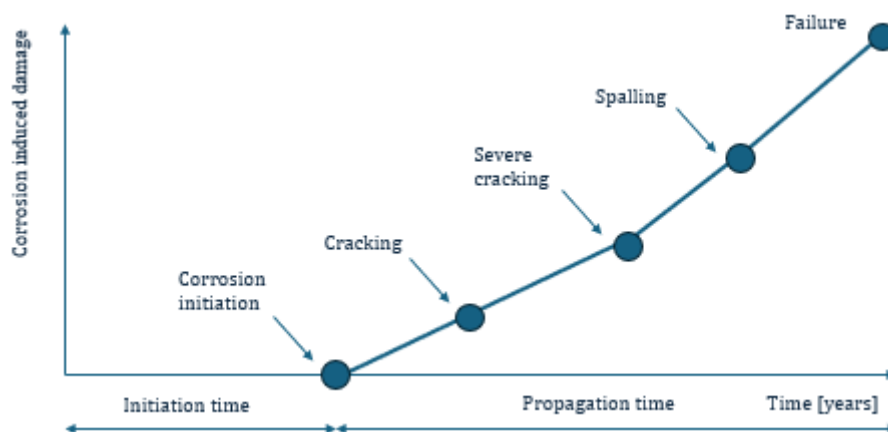


Figure 2.1. Service life of RC structures in the presence of chloride-induced corrosion.

### 2.2.2 Chloride-Induced Corrosion

Modeling chloride-induced corrosion in RC elements requires models to estimate the chloride ions concentration in the concrete matrix over time. A number of models are available to accomplish this task. Considering the unidimensional chloride diffusion process, Fick's second Law is commonly used to model the time variant chloride concentration at the depth of the reinforcement layer. Given the inherent uncertainty associated with the chloride diffusion process, a probabilistic approach is necessary. A significative research effort led to the development of the DuraCrete model [7], which allowed to include the uncertainty that affects all the parameters involved in the corrosion process using a simulation approach. The model represented also the basis for the *fib* model [8][9] which is based on the error function solution of Fick's 1D law of diffusion. The mathematical model is described by Equation (2.1):

$$(2.1) \quad C(x, t) = C_0 + (C_{s, \Delta x} - C_0) \cdot \left[ 1 - \operatorname{erf} \left( \frac{x - \Delta x}{2 \cdot \sqrt{D_{app}(t) \cdot t}} \right) \right]$$

In Equation (2.1)  $C_0$  is the initial chloride content,  $C_{s, \Delta x}$  is the chloride content resulting from environmental exposure at depth  $\Delta x$ ,  $\Delta x$  is the depth of the convection zone (which is the superficial concrete layer up to which chloride penetration is not controlled by Fick's second law of diffusion),  $C(x, t)$  is the chloride content at depth  $x$  at the time  $t$ ,  $\operatorname{erf}$  is the Gaussian error function and  $D_{app}(t)$  is the apparent chloride diffusion coefficient at time  $t$ .  $D_{app}(t)$  can be computed according to Equation (2.2):

$$(2.2) \quad D_{app}(t) = k_e \cdot D_{rcm,0}(t_0) \cdot k_t \cdot A(t)$$

Where  $D_{rcm,0}(t_0)$  is the chloride diffusion coefficient at reference time  $t_0$ ,  $k_e$  is the environmental transfer variable,  $k_t$  is the transfer variable and  $A(t)$  is the aging parameter.  $k_e$  is defined as per Equation (2.3):

$$(2.3) \quad k_e = \exp \left( b_e \left( \frac{1}{T_{ref}} - \frac{1}{T_{real}} \right) \right)$$

In Equation (2.3),  $b_e$  is the temperature coefficient,  $T_{ref}$  is the reference temperature and  $T_{real}$  is the temperature of the structural element.  $A(t)$  is defined as per Equation (2.4):

$$(2.4) \quad A(t) = \left( \frac{t_0}{t} \right)^a$$

In Equation (2.4),  $a$  is the aging exponent and  $t_0$  is the reference time taken as 28 days. It is worth noting that every variable involved in the chloride diffusion process is usually

modeled as a random variable (RV) with a given distribution. Reference distributions can be retrieved in [8][9].

In order to apply the probabilistic approach to the identification of chloride-induced corrosion initiation time  $t_i$ , it is necessary to refer to a limit state function  $g$ . In general, a limit state function identifies a condition which represents the contrast between actions  $S$  and resistance  $R$ . Considering that  $S$  and  $R$  are generally RVs and that their value may be time dependent, a limit state function  $g(X, t)$  is a RV that can be defined as per Equation (2.5):

$$(2.5) \quad g(X, t) = R(t) - S(t)$$

In case of chloride-induced corrosion, the limit state function is specified as per Equation (2.6):

$$(2.6) \quad g(X, t) = C_{crit} - C(x, t)$$

where  $C_{crit}$  is the critical chloride content threshold. Corrosion is assumed to start when  $g(X, t) \leq 0$ , indicating that the critical chloride content threshold has been reached at the depth of the reinforcement layer. The probability of corrosion initiation can be computed from Equation (2.7):

$$(2.7) \quad P[g(X, t) \leq 0] = P[C_{crit} - C(x, t) \leq 0]$$

One important aspect of the process is represented by the  $C_{crit}$  value, which is one the controlling parameters in the estimation of  $t_i$  and is affected by a significative variability in literature data. In [9], a mean value of 0.6% of binder weight is reported. The work of Angst et al. [10] summarizes the factors influencing the  $C_{crit}$  value, highlighting the significative discrepancy in the values reported in literature, with values ranging from 0.04 to 8.34% of binder weight. The review revealed that available results may have been influenced by test setup, that binders different from the ordinary Portland cement are characterized by contradictory results and that the most important factors affecting  $C_{crit}$  are the steel-concrete interface and the steel potential. Hussain and Al-Gahtani [11] reported  $C_{crit}$  values from 0.22% to 0.29% of binder weight. Alonso et al. [12] pointed out that the scatter between the  $C_{crit}$  values reported in the literature is due to the large number of variables involved and that a uniform definition of  $C_{crit}$  was missing. The  $C_{crit}$  values found in the study range from 0.39% to 1.16% of binder weight. Polder [13] studied the relationship between  $C_{crit}$  and concrete electrical resistivity. Theoretically, low concrete resistivity values should be related to low  $C_{crit}$  values, but experimental data only partially support this. Angst et al. [14] studied the dependency of  $C_{crit}$  from the specimen size and pointed out the poor accuracy of test methods. The work of Val [15] also pointed out the discrepancy affecting  $C_{crit}$  values definition in literature.



The effects of different values of  $C_{crit}$  on corrosion initiation can be demonstrated through the following exercise. The chloride content over time for a given exposure condition has been computed assuming 50.5 mm cover depth and using Equation (2.1). For the sake of comparison, only the mean values of the parameters retrieved from [9] have been used. It is evident that, assuming that corrosion initiates once that  $C_{crit}$  is reached on the external surface of steel reinforcement, the discrepancies in the estimation of  $C_{crit}$  may lead to significant uncertainty in predictive modeling of RC deterioration mechanisms. Table 2.1 reports the values of corrosion initiation time for different mean values of  $C_{crit}$ . The minimum and maximum values of corrosion initiation time are 2.1 and 27.1 years, confirming the uncertainty observed in the literature. The  $C_{crit}$  value suggested in [8][9] represents an intermediate result, being associated with corrosion initiation after 11.2 years.

**Table 2.1. Corrosion initiation time for different mean values of  $C_{crit}$ .**

| $C_{crit}$ [wt.-%b] | Corrosion Initiation Time [years] |
|---------------------|-----------------------------------|
| 0.04                | 2.1                               |
| 0.22                | 4.9                               |
| 0.29                | 6.0                               |
| 0.39                | 7.5                               |
| 0.60                | 11.2                              |
| 1.16                | 27.1                              |

### 2.2.3 Carbonation-Induced Corrosion

Modeling the effects of carbonation-induced corrosion requires to estimate the evolution of the depth of the carbonation front over time. Amongst the models available in the literature, the one developed by *fib* is commonly used [8]. The model was first developed within the framework of the DuraCrete project [7] and later revised during the activities of the DARTS project [16]. The model uses Equation (2.8) to estimate the carbonation front depth  $x_c(t)$ :

$$(2.8) \quad x_c(t) = \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot C_s \cdot \sqrt{t} \cdot W(t)}$$

In Equation (2.8),  $k_e$  is the environmental function,  $k_c$  is the execution transfer parameter,  $k_t$  is the regression parameter,  $R_{ACC,0}^{-1}$  is the inverse effective carbonation resistance of concrete,  $\varepsilon_t$  is the error term,  $C_s$  is the  $CO_2$  concentration and  $W(t)$  is the weather function. Note that Equation (2.8) is based on the assumption that diffusion is the prevailing

mechanism in CO<sub>2</sub> penetration in the concrete matrix and that the diffusion coefficient of CO<sub>2</sub> in the concrete matrix is constant over time.  $k_e$  is defined according to Equation (2.9):

$$(2.9) \quad k_e = \left( \frac{1 - \left( \frac{RH_{real}}{100} \right)^{f_e}}{1 - \left( \frac{RH_{ref}}{100} \right)^{f_e}} \right)^{g_e}$$

Where  $RH_{real}$  is the relative humidity of the carbonated layer,  $RH_{ref}$  is the reference relative humidity,  $f_e$  and  $g_e$  are exponents determined according to experimental data available in literature.  $k_e$  is defined as per Equation (2.10):

$$(2.10) \quad k_c = \left( \frac{t_c}{7} \right)^{b_c}$$

In Equation (2.10),  $t_c$  is the period of curing and  $b_c$  is a regression exponent. The values of  $k_e$  and  $\varepsilon_e$  depend on the testing method. Reference values of  $R^{-1}_{ACC,0}$  according to [8] are reported in Table 2.2.  $C_s$  can be computed according to Equation (2.11)

$$(2.11) \quad C_s = C_{s,atm} + C_{s,emi}$$

In Equation (2.11),  $C_{s,atm}$  is the atmospheric CO<sub>2</sub> concentration and  $C_{s,emi}$  is the additional CO<sub>2</sub> concentration due to emission sources, usually neglected. Finally,  $W(t)$  can be computed according to the meso-climatic conditions as per Equation (2.12):

$$(2.12) \quad W = \left( \frac{t_0}{t} \right)^w$$

In Equation (2.12),  $t_0$  is the reference time [years] and  $w$  is the weather exponent. The statistical distributions that can be used to model the RVs involved in the carbonation process can be retrieved in [8].

In case of carbonation-induced corrosion, it is assumed that corrosion starts once that the carbonation front reaches the reinforcement depth. The limit state function to estimate corrosion initiation is specified as per Equation (2.13):

$$(2.13) \quad g(X, t) = x - x_c(t)$$

Corrosion is assumed to start when  $g(X, t) \leq 0$ , indicating that the carbonation has reached the depth  $x$  of the reinforcement layer. The probability of corrosion initiation can be computed from Equation (2.14):

$$(2.14) \quad P[g(X, t) \leq 0] = P[x - x_c(t) \leq 0]$$

Table 2.2. Values of  $R^{-1}_{ACC,0}$  according to [8] in  $[10^{-11} \text{ (m}^2/\text{s)}/(\text{kg}/\text{m}^3)]$

|                           | w/c  |      |      |      |      |      |
|---------------------------|------|------|------|------|------|------|
| Cement type               | 0.35 | 0.40 | 0.45 | 0.50 | 0.55 | 0.60 |
| CEM I 42.5 R              | -    | 3.1  | 5.2  | 6.8  | 9.8  | 13.4 |
| CEM I 42.5 R + FA (k=0.5) | -    | 0.3  | 1.9  | 2.4  | 6.5  | 8.3  |
| CEM I 42.5 R+ SF (k=2.0)  | 3.5  | 5.5  | -    | -    | 16.5 | -    |
| CEM III/B 42.5            | -    | 8.3  | 16.9 | 26.6 | 44.3 | 80.0 |

#### 2.2.4 Corrosion-induced Concrete Cracking in Corroded RC Elements

As previously mentioned, the expansion of corrosion products over time leads to cover concrete cracking. Over the years, a number of models have been developed to address this issue. Bazant [17] proposed a model based on the thick wall cylinder idealization. Andrade et al. [18][19] conducted an experimental campaign and developed a model based on the numerical simulation of the cracking induced by corrosion in RC specimens. Vu et al. [20] conducted an experimental campaign on eight RC column specimens and proposed an empirical model to predict time to excessive cracking in corroded RC structures. They also observed that the prediction of cracking initiation does not affect significantly the time to excessive cracking. Bhargava et al. [21] proposed an analytical model to compute the time to cracking initiation based on a boundary problem idealization and then validated it through comparison with experimental results. El Maaddawy and Soudki [22] proposed a mathematical model connecting steel mass loss and radial pressure caused by expansion of corrosion products to estimate cracking initiation time. The base of the model was the thick-walled cylinder idealization and concrete is assumed to crack when the circumferential tensile stresses in concrete reach the tensile strength. Alternative approaches to address the problem involved fracture mechanics [23] and plastic analysis [24].

One of the main parameters influencing corrosion-induced cracking is the rust expansion coefficient. Zhao et al. [25] proposed values ranging from 2.6 to 3.3 depending on the environmental conditions, based on experimental data obtained from eight corrosion products collected from different environments. Caré et al. [26] performed accelerated

corrosion tests and studied the resulting corrosion pattern through Digital Image Correlation, studying the expansion of corrosion products and giving recommendations to estimate rust's Young modulus.

Andrade et al. [27] proposed expressions to estimate corrosion-induced crack width, relating corrosion penetration, cover depth and bar diameter. Liang et al. [28] applied a number of models exploring the service life prediction for a bridge structure, revealing a good agreement between the results obtained by applying different models. A study on the service life of RC structures [29] pointed out that the main factors affecting cracking time are cover depth, concrete material properties and rust product accumulation. Vidal et al. [30] introduced a relationship between reinforcement corrosion and crack width based on experimental data. The linear relationship between crack width  $w(t)$  and corrosion is described by Equation (2.15):

$$(2.15) \quad w(t) = K(\Delta_s(t))$$

In Equation (2.15),  $K$  is a factor equal to 0.0575 and  $\Delta_s$  is the difference between uncorroded bar area and bar area at time  $t$ .

### 2.2.5 Corrosion Rate

Once that corrosion initiates, the propagation phase (and the associated propagation period  $t_p$ ), goes from corrosion initiation to the end of the service life of a structure, which is generally associated with thresholds in terms of mass loss or decrease of structural performance.

The corrosion propagation phase can be described using Faraday's law [31]:

$$(2.16) \quad p(t_p) = 0.0116 \cdot R \cdot t_p \cdot i_{corr}$$

In Equation (2.16),  $R$  is the pitting factor,  $t_p$  is the propagation period and  $i_{corr}$  is the corrosion current density in  $\mu\text{A}/\text{cm}^2$ . The factor 0.0116 is present to convert  $\mu\text{A}/\text{cm}^2$  into mm/year. The value of  $R$  depends on the morphology of corrosion, which can be modeled as uniform or localized (pitting).

Corrosion rate is one of the main factors controlling the propagation phase and the reinforcement area loss. Over the years, a number of studies have tried to characterize this value, which is influenced by several parameters. Vu and Stewart [32] pointed out that corrosion rate is governed by the availability of water and oxygen at the steel surface, implying a dependence from cover and concrete quality. They also observed that in case of high relative humidity (like many locations in Australia, US and Europe) corrosion rate is controlled by oxygen availability at the cathode, while in case of low relative humidity the

controlling factor is the concrete electrical resistivity [33][34][35]. The study proposed to estimate corrosion current density at the start of the propagation phase  $i_{corr}(1)$  as per Equation (2.17):

$$(2.17) \quad i_{corr}(1) = \frac{37.8 \cdot \left(1 - \frac{w}{c}\right)^{-1.64}}{x}$$

where  $w/c$  is the water to cement ratio of the concrete. Corrosion rate is not constant over time since it is expected that the formation of rust products on the steel surface will reduce the diffusion of the iron ions away from the steel surface. The expected reduction of area ratio between anode and cathode is also expected to contribute to a reduction of the corrosion rate over the years. Given this evidence, the authors of the study used the data reported in [36] to relate  $t_p$  and  $i_{corr}$  as per Equation (2.18):

$$(2.18) \quad i_{corr}(t_p) = 0.85 \cdot i_{corr}(1) \cdot t_p^{-0.29}$$

The initial value of  $i_{corr}$  for various  $w/c$  and cover depth values, and the normalized corrosion rate  $i_{corr}(t_p)/i_{corr}(1)$  are showed in Figure 2.2. The model captures the fact that the corrosion rate reduces rapidly during the first few years after initiation but then more slowly as it approaches a nearly uniform level. Other predictive models have been developed over the decades. Liu and Weyers [36] proposed a nonlinear regression model based on 2927 test results that connected corrosion rate with chloride content, ambient temperature, concrete ohmic resistance and active corrosion duration. Morinaga [37] proposed an empirical corrosion rate model depending on chloride content, ambient temperature, oxygen concentration and relative humidity. Kong et al. [38] proposed a model based on [36] in which the corrosion rate variation depends on chloride content, ambient temperature and concrete resistivity. In some cases [39][40] corrosion rate evolution over time is not considered or is considered neglecting the factors influencing corrosion, undermining the applicability of models. Pour-Ghaz et al. [41] developed a closed form regression model based on the numerical solution of the Laplace equation with predefined boundary conditions [41]. These empirical models have been compared with experimental data in [42], where a new model has also been proposed including the influence of chloride content, ambient temperature, relative humidity, concrete resistivity and corrosion time.

The factors affecting corrosion rate have been extensively studied, and a summary is provided in Table 2.3. Alonso et al. [43] observed a strong correlation between corrosion rate and electrical resistance of concrete, deducing that concrete electrical resistivity may act as a controlling factor of the corrosion rate. Lopez and Gonzalez [44] correlated corrosion rate, concrete resistivity and degree of pore saturation. They observed that the degree of pore saturation strongly influences the corrosion kinetics and determines the active state of the corrosion process. Balabanić et al. [45] analyzed the influence of concrete

quality (in terms of  $w/c$ ), cover thickness and degree of water saturation on the corrosion current density. The results revealed that, if other parameters are constant, concrete quality is more important than concrete cover depth on the value of corrosion current density. The work of Yalcin and Ergun [33] studied the corrosion rate evolution over time, identifying an exponential decay due to the drying of concrete and proposing a predictive equation. Alonso et al. [46] extensively studied the influence of cover/bar diameter, proportions of cement,  $w/c$ , bar position, transverse reinforcement and corrosion rate on concrete cracking. They observed that corrosion rate is a key factor in crack development. Takewaka et al. [47] proposed a corrosion rate simulation model based on the supply rate of oxygen, necessary for the reaction in the cathode area. Gulikers [48] studied the electrochemical process underlying corrosion from the theoretical point of view, highlighting that a linear relationship between corrosion current density and concrete resistivity does not necessarily imply that concrete resistance is dominating the overall corrosion cell resistance. Also, two corrosion rate models have been proposed. Otieno et al. [49] critically reviewed the available corrosion rate prediction models and presented results oriented towards the development of an improved corrosion rate model using both natural and accelerated corrosion results.

### 2.2.6 Uniform and Localized Corrosion

Among the effects of corrosion on RC structures, reinforcement section loss is one of the most obvious effects. Corrosion on steel reinforcement can be uniformly distributed around the bar or can be localized (pitting corrosion). Reinforcement diameter loss is described using Faraday's Law (Equation (2.16)) and the pitting factor  $R$  is the controlling factor to define the corrosion morphology. The value of  $R$  for uniform corrosion is equal to 2. The uniform corrosion model is simple and computationally convenient when running nonlinear analysis [50][51]. However, field data [51] revealed that pitting corrosion is more relevant for chloride-induced corrosion. The residual area of a corroded bar obtain applying the uniform corrosion model  $A_C$  can be consequently computed as per Equation (2.19) [52][53]:

$$(2.19) \quad A_C(t) = \begin{cases} \frac{\pi D_0^2}{4} & \text{for } t < t_i \\ \frac{\pi [D_0 - 2 \int_{t_i}^t 0.0116 \cdot i_{corr}(t_p) dt_p]^2}{4} & \text{for } t > t_i \end{cases}$$

Where  $D_0$  is the rebar diameter in uncorroded condition.

Literature studies [54] revealed that chloride-induced corrosion of steel reinforcement starts with the formation on pits along the length of the bar. Then, as the number of pits increases, they tend to join up and cause uniform corrosion. Pitting corrosion can be described as a form of localized corrosion which causes significant section loss in

reinforcing bars. In pitting corrosion, the formation of macrocell (small anodic area compared with large cathodic area) has been observed in the literature [55][56]. More details on the nature of the chemical reaction occurring in case of pitting corrosion can be found in [57]-[59].

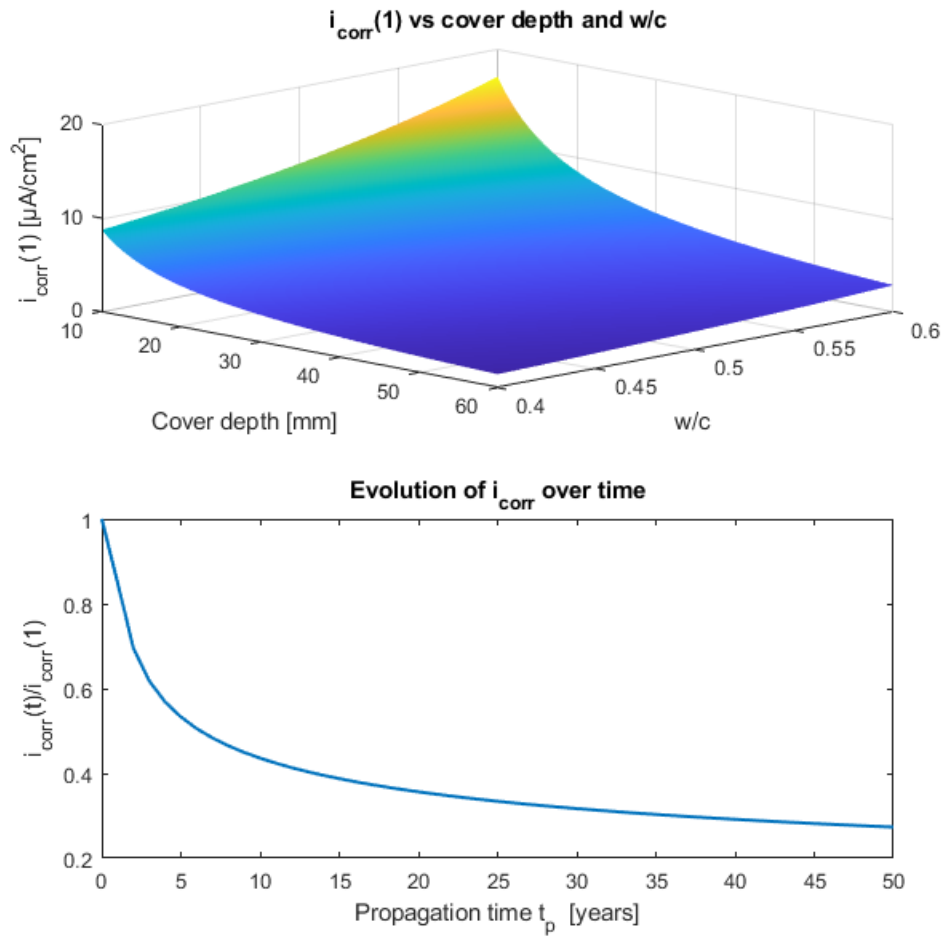


Figure 2.2. Initial value of  $i_{\text{corr}}$  for varying w/c and cover depth values and normalized values of  $i_{\text{corr}}$  over time.

Table 2.3. Main factors controlling the corrosion rate.

| Reference               | Factors controlling the corrosion rate                                                            |
|-------------------------|---------------------------------------------------------------------------------------------------|
| Vu and Stewart [32]     | Availability of water and oxygen at the steel surface, cover depth and concrete quality           |
| Liu and Weyers [36]     | Chloride content, ambient temperature, concrete ohmic resistance and active corrosion duration    |
| Morinaga [37]           | Chloride content, ambient temperature, oxygen concentration and relative humidity                 |
| Kong et al. [38]        | Chloride content, ambient temperature and concrete resistivity                                    |
| Lu et al. [42]          | Chloride content, ambient temperature, relative humidity, concrete resistivity and corrosion time |
| Alonso et al. [43]      | Concrete electrical resistivity                                                                   |
| Lopez and Gonzalez [44] | Concrete resistivity and degree of pore saturation                                                |
| Balabanić et al. [45]   | Concrete quality                                                                                  |
| Yalcin and Ergun [33]   | Progressive drying of concrete                                                                    |

$R$  in Equation (2.16) allows to compute the maximum pit depth in a reinforcement bar. Usual values of  $R$  range from 4 to 8 for natural corrosion and 5 to 13 for accelerated corrosion techniques [52]. Assuming hemispherical pit morphology, according to Val's model [31][60], the residual area in presence of pitting  $A_{R,P}$  and the pit width  $a(t)$  can be estimated as per Equations (2.20)-(2.21):

$$(2.20) \quad a(t) = 2p(t) \sqrt{1 - \left(\frac{p(t)}{D_0}\right)^2}$$

$$(2.21) \quad A_{R,P} = \begin{cases} \frac{\pi D_0^2}{4} - A_1 - A_2 & \text{for } p(t) \leq \frac{\sqrt{2}}{2} D_0 \\ A_1 - A_2 & \text{for } \frac{\sqrt{2}}{2} D_0 < p(t) \leq D_0 \\ 0 & \text{for } p(t) > D_0 \end{cases}$$

where:



$$(2.22) \quad A_1 = \frac{1}{2} \left[ \theta_1 \left( \frac{D_0}{2} \right)^2 - a(t) \left| \frac{D_0}{2} - \frac{p(t)^2}{D_0} \right| \right]$$

$$(2.23) \quad A_2 = \frac{1}{2} \left[ \theta_2 \cdot p(t)^2 - a \cdot \frac{p(t)^2}{D_0} \right]$$

$$(2.24) \quad \theta_1 = 2 \arcsin \left[ \frac{a(t)}{D_0} \right]$$

$$(2.25) \quad \theta_2 = 2 \arcsin \left[ \frac{a(t)}{2p(t)} \right]$$

Finally, the residual area in a given pit location  $A_{C,P}$  can be estimated according to [54][61]:

$$(2.26) \quad A_{C,P} = \left[ 1 - \frac{a(t)}{2D_0} \right] [A_{C,U}(t) - A_0] + A_{R,P}(t)$$

where  $A_0$  is the uncorroded bar area. Figure 2.3 graphically represents uniform and pitting corrosion models. The value of  $R$  is affected by a huge variability in literature and robust pitting factors are needed [53]. Gonzalez et al. [52] compared average corrosion values with maximum pit depth values using natural and accelerated corrosion tests and found that the maximum penetration of localized attack on steel embedded in concrete containing chlorides is equivalent to about four to eight times the average general penetration. Apostolopoulos et al. [62] studied bare and embedded B500C steel bars and found that embedded bars were affected by more severe pitting corrosion. The inherent variability of pitting corrosion distribution requires characterizing  $R$  as a statistical distribution, and considerable research effort has been devoted to this matter. Andisheh et al. [63] conducted a systematic survey of the literature and investigated the correlation between  $D_0$  and  $R$ . The relationship between  $R$  and  $D_0$  [mm] can be expressed according to Equation (2.27):

$$(2.27) \quad R = 0.0706D_0 + 7.0538$$

Table 2.4 reports information on the  $R$  factors in literature [64]-[70].

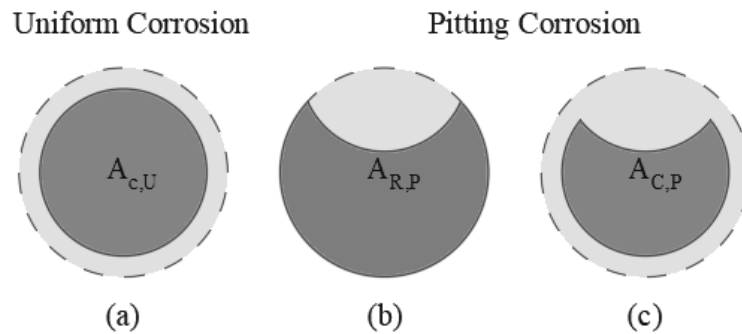


**Figure 2.3.** (a) Uniform Corrosion model, (b) Pit formation on the rebar, (c) as the number of pits increases, they tend to join causing uniform corrosion.

Table 2.4. Pitting Factors available in literature.

| Reference                             | $D_o$ [mm] | $R$ (mean) | $R$ (COV) |
|---------------------------------------|------------|------------|-----------|
| Torres-Acosta and Martínez-Madrid[64] | 13         | 5.5        | 0.59      |
| Stewart [65]                          | 10         | 5.62       | 0.22      |
|                                       | 16         | 6.20       | 0.18      |
|                                       | 27         | 7.10       | 0.17      |
| Rodriguez et al. [66]                 | 12         | 4.00       | 0.15      |
| Stewart [67]                          | 8          | 7.00       | 0.18      |
|                                       | 16         | 7.68       | 0.16      |
|                                       | 24         | 8.08       | 0.16      |
|                                       | 28         | 8.23       | 0.15      |
|                                       | 32         | 8.36       | 0.15      |
|                                       | 36         | 8.48       | 0.15      |
| Cairns et al. [68]                    | 16         | 23.80      | 0.56      |
| Torres-Acosta et al. [69]             | 10         | 11.70      | -         |
| Stewart and Al-Harthy [70]            | 16         | 6.20       | 0.18      |
|                                       | 27         | 7.10       | 0.17      |

### 2.2.7 Influence of Concrete Cracking on Corrosion Initiation and Propagation

The models currently used in the field of life cycle assessment of RC structures do not include the influence of cracks on corrosion initiation. Cracks represent preferential routes for aggressive agents such as chlorides, influencing significantly the evolution of corrosion over time. Considering that RC structures crack under service loads, current models are less robust than expected. The effect of cracks on corrosion initiation and propagation has been studied extensively over the years, both experimentally and numerically, in order to try to overcome the limitations of current corrosion models. In uncracked concrete, a passive layer on the steel surface prevents corrosion which starts after its destruction. The presence of cracks allows chloride to penetrate rapidly, and the passive film is never produced [71]. Cracks affect significantly corrosion initiation, but no relationship between crack width and critical chloride content  $C_{crit}$  has been found. The influence of cracks on corrosion propagation and mitigation effects are also open debates [71]. The observation that cracks may lead to accelerating the corrosion process with respect to uncracked concrete is typical of laboratory studies, not including crack healing effects that may occur over time and reduce corrosion rate. Also, from the chemical point of view, the kinetics of corrosion in cracked concrete are controlled by the oxygen and moisture transport in the

uncracked cover concrete, and thus the corrosion rate may be similar to corrosion in uncracked concrete [72].

Lu et al. [73] proposed a reliability-based approach to predict  $t_i$  in RC cracked beams exposed to de-icing salts and developed a model to estimate chloride diffusivity in cracked concrete based on the Eurocode 2 model for cracks [74] and on the chloride diffusion coefficient inside the crack proposed by Djerbi et al. [75]. The study demonstrated that cracks should be considered to estimate correctly  $t_i$  in RC elements exposed to chlorides. An experimental campaign conducted by Yu et al. [76] revealed that exposure conditions and cover depth influence corrosion evolution over time in cracked RC beams exposed to chlorides. Otieno et al. [77][78] conducted an extensive experimental campaign on naturally and artificially corroded beams investigating corrosion propagation in marine environments and proposed two corrosion rate models. The study revealed that field performance under natural corrosion cannot be inferred from laboratory performance under accelerated corrosion. The influence of crack width, cover depth, binder type and  $w/b$  ratio on corrosion rate  $i_{corr}$  have also been investigated, highlighting that  $i_{corr}$  increases as crack width increases and decreases as concrete quality and cover depth increase. It has been pointed out that the reported corrosion rates were averaged over the whole steel surface, but the assumption leads to underestimating corrosion rates and penetration depth. In cracked concrete this is particularly important since high local pitting is expected in proximity of the cracks. Also, even if the proposed models for  $i_{corr}$  match well the experimental data, independent data are needed to assess the accuracy of the models.

The work of Kathler et al. [79] reports that, although there is an agreement on the fact that cracking accelerates chloride ingress and corrosion initiation, data regarding the influence of cracks on corrosion propagation are contradictory and that the reason might be found in differences in test and exposure conditions and in the lack of time-resolved data in cyclic-wetting exposure. An experimental campaign to investigate the influence of cyclic wetting-drying exposure in the presence of cracked concrete has been conducted and a model to estimate  $i_{corr}$  accounting for that with an exponential decay over time has been developed. Jones et al. [80] simulated the effects of crack-filling materials to repair cracks in concrete and to delay corrosion initiation. The study revealed that the presence of cracks allows chloride to bypass the bulk concrete, reducing significantly the effectiveness of concrete cover depth and that the effectiveness of repair strategies is strictly related to their impact on the diffusivity of the damaged zone. Leung and Hou [81] carried out an extensive numerical analysis that resulted in the empirical equation of an equivalent diffusion coefficient  $D_{eq}$  including the effect of concrete cracking applicable to model chloride-induced corrosion initiation and propagation. The proposed  $D_{eq}$  implicitly includes the diffuence effect in the crack and is accurate for typical concrete cover depth of RC elements. The study of Otieno et al. [82], based on an experimental campaign on cracked

beam specimens, introduced the concept of performance-based crack width limits, since a universal value is not valid for a wide range of exposure conditions and cover depths. Bezuidenhout and van Zijl [83] studied experimentally the influence of multiple cracks and reinforcement layouts on the corrosion process and proposed a model to estimate the residual service life of a structure that entered the propagation phase. Russo et al. [84] studied the corrosion of carbon and stainless-steel bars, confirming the discrepancies that can be found in the literature regarding corrosion propagation in presence of cracks. The experimental campaign showed that microcracks are fundamental in accelerating corrosion phenomena in carbon steel rebars, while stainless steel rebars are still in passive conditions even in presence of cracked concrete. O'Reilly et al. [85] highlighted that even narrow cracks (0.025 to 0.1 mm) can lead to a sensible reduction of  $t_i$ . Zhang et al. [86] investigated how environmental humidity, load-induced crack and stress state affect the chloride diffusion process in concrete, studying corrosion initiation with a probabilistic approach.

### 2.3 CORROSION INDUCED DETERIORATION IN RC ELEMENTS

Besides the reinforcement area reduction, corrosion modifies the mechanical properties of steel material, induces cracks that reduce concrete cover compressive strength, weakens the bond between steel and concrete and affects the confined concrete properties. This section summarizes the main effects of corrosion on these aspects, highlighting the main experimental and numerical findings and the possible approaches that can be applied to include corrosion-induced effects in the simulation of RC elements.

#### 2.3.1 Steel Tensile Response

A significative number of studies investigated the tensile response of corroded steel bars. Du et al. [87][88] studied the effects of corrosion on capacity and ductility of corroded bars through tests on bare and embedded steel bars and found that corrosion alters the shape of the rebar and that residual yield and ultimate strength decrease more rapidly than bar section. They also pointed out that bar diameter does not influence the capacity reduction due to corrosion and that steel ductility significantly decreases with corrosion (much more than yield and ultimate strength, as represented in Figure 2.4), while strength ratio, hardening modulus and hardening strain are not affected by corrosion. The authors proposed some relations to estimate the time dependent material properties of steel bars:

$$(2.28) \quad f_y(t) = f_y(0) \cdot (1 - \alpha_{fy} \cdot \psi)$$

$$(2.29) \quad f_u(t) = f_u(0) \cdot (1 - \alpha_{fu} \cdot \psi)$$

$$(2.30) \quad \varepsilon_{su}(t) = \varepsilon_{su}(0) \cdot (1 - \alpha_\varepsilon \cdot \psi)$$

In Equations (2.28)-(2.30),  $f_y$  is the steel yield strength,  $f_u$  is the steel ultimate strength,  $\varepsilon_{su}$  is the steel ultimate strain,  $\psi$  is the bar section loss percentage and  $\alpha_{fy}$ ,  $\alpha_{fu}$  and  $\alpha_\varepsilon$  are empirical coefficients.

Lee and Cho [89] investigated experimentally steel tensile strength at different stages of reinforcement corrosion and noted a progressive decrease of  $f_y$  and nominal elastic modulus as corrosion progressed. The reduction factors for pitting corrosion have been found to be more severe than for uniform corrosion due to the more severe localized section loss in pitted sections. Apostolopoulos and Papadopoulos [90] tested the degradation of the tensile properties of steel bars using an accelerated corrosion technique. The tests revealed that material deterioration may be more severe than expected by taking mass loss as a damage measure. The results confirm that ductility deterioration is more severe than strength deterioration and that ductility reduction impacts also the low-cycle fatigue behavior of the steel bars.

Hawileh et al. [91] studied monotonic and low-cycle fatigue of steel bars considering three different corrosion levels, confirming that corrosion is more impactful on ductility and low-cycle fatigue than on strength. Zhang et al. [92] conducted natural and artificial corrosion tests investigating tensile and low-cycle fatigue response of steel bars. They found that fatigue life decreases more significantly than tensile response with corrosion and that artificially corroded rebars are less affected by corrosion than naturally corroded bars. Cairns et al. [93] conducted experimental and numerical studies on the effects of corrosion on reinforcement mechanical properties. Oyado et al. [94] tested 13 RC members and measured corrosion on bars through a 3D scanning technique. The authors found that the tensile strength of the bars was not decreased only by the average cross section loss, but also by localized corrosion effects. Apostolopoulos et al. [95] analyzed the effects of corrosion exposure on steel bars and pointed out that corrosion may lead steel bars mechanical properties to decrease below the code-specified limits. Alexopoulos et al. [96] tested steel bars produced by different manufactures and introduced the concept of a performance index combining tensile strength and ductility. Andisheh et al. [53][97] investigated the effects of pitting corrosion experimentally, proposed a time dependent pit depth model and, starting with the consideration that reduction factors reported in literature are affected by large scatter, proposed a methodology to developed advanced reduction factors for the mechanical properties of corroded steel bars.

Given the inherent variability in the modification of steel mechanical properties due to corrosion, Table 2.5 reports the  $a_{fy}$ ,  $a_{fu}$  and  $a_e$  identified through the experimental research carried out through the years. On one hand it is evident how each reduction factor is characterized by significant variability, while on the other hand a general trend confirming that ductility is more affected than strength by corrosion damage has been found. While it is almost impossible to summarize all the studies available in the literature regarding the tensile response of corroded steel bars, the interested reader may refer to [98]-[109].

**Table 2.5. Empirical coefficients for steel mechanical properties deterioration based on the available literature.**

| Reference                                     | $D_0$<br>[mm]                             | $\psi$<br>[%]          | Corrosion<br>Method                  | $\alpha_{fy}$                              | $\alpha_{fu}$                              | $\alpha_\varepsilon$                              |
|-----------------------------------------------|-------------------------------------------|------------------------|--------------------------------------|--------------------------------------------|--------------------------------------------|---------------------------------------------------|
| Du et al.<br>[87][88]                         | 8, 16, 32<br>(Ribbed)<br>8, 16<br>(Plain) | 0-25                   | Accelerated,<br>simulated            | 0.0048                                     | 0.0061                                     | 0.03 (Bare<br>bars)<br>0.05<br>(Embedded<br>bars) |
| Lee and Cho<br>[89]                           | 10<br>13                                  | 0-35                   | Accelerated,<br>chloride-<br>induced | 0.0124<br>(Uniform)<br>0.0198<br>(Pitting) | 0.0107<br>(Uniform)<br>0.0157<br>(Pitting) | 0.0195<br>(Uniform)<br>0.0259<br>(Pitting)        |
| Apostolopoulos<br>and<br>Papadopoulos<br>[90] | 10                                        | 0-<br>8.48             | Accelerated<br>(salt spray)          | 0.0043                                     | 0.0028                                     | 0.051                                             |
| Hawileh et al.<br>[91]                        | 10                                        | 0-<br>19.60            | Accelerated                          | 0.0144                                     | 0.0136                                     | 0.0418                                            |
| Zhang et al.<br>[92]                          | 6.5<br>12                                 | 14-38<br>4-28          | Natural<br>Accelerated               | 0.0112<br>0.0110                           | 0.0136<br>0.0122                           | -<br>-                                            |
| Cairns et al. [93]                            | 16, 20                                    | 0-3                    | Accelerated                          | 0.012                                      | 0.011                                      | 0.03                                              |
| Apostolopoulos<br>et al. [95]                 | 8                                         | 0-<br>31.52            | Accelerated<br>(salt spray)          | 0.0092                                     | 0.0064                                     | 0.0163                                            |
| Andisheh et al.<br>[97]                       | 10, 12,<br>16                             | 0-10<br>10-20<br>20-30 | Accelerated                          | 0.0151<br>0.0139<br>0.0134                 | 0.0166<br>0.0158<br>0.0152                 | 0.032<br>0.026<br>0.021                           |
| Andrade et al.<br>[98]                        | 12                                        | 0-11                   | Accelerated                          | 0.015                                      | 0.013                                      | 0.017                                             |
| Morinaga [99]                                 | -                                         | 0-25                   | Natural                              | 0.017                                      | 0.018                                      | 0.06                                              |
| Castel et al.<br>[100]                        | 6, 12                                     | 0-20                   | Environmental<br>Exposure            | -                                          | -                                          | 0.035                                             |
| Du [101]                                      | 8, 16, 32                                 | 0-25<br>0.18           | Accelerated                          | 0.014<br>0.015                             | 0.014<br>0.015                             | 0.029<br>0.039                                    |
| Almusallam<br>[102]                           | 6<br>12                                   | 0-80                   | Accelerated                          | -<br>-                                     | 0.0117<br>0.011                            | -<br>-                                            |
| Lee et al. [103]                              | -                                         | 0-25                   | Accelerated                          | 0.012                                      | -                                          | -                                                 |
| Saifullah and<br>Clark [104]                  | -                                         | 0-28                   | Accelerated                          | 0.013<br>0.012                             | 0.017<br>0.014                             | -<br>-                                            |

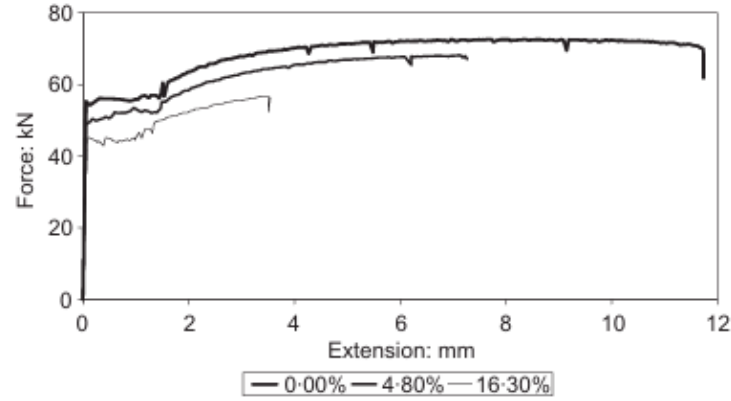


Figure 2.4. Tensile response of steel bars with different levels of reinforcement, from Du et al. [87]

### 2.3.2 Low-cycle Fatigue and Buckling of Reinforcing Bars

As previously mentioned, corrosion not only influences the steel monotonic response, but also affects fatigue life and buckling response of reinforcing bars. Apostolopoulos [110] studied experimentally the accumulated damage on corroded steel bars. The details of the test program are reported in the study, with three cycles at 1%, 2.5% and 4% strain for several corrosion levels. The test results revealed that fatigue life, which can be predicted through the Coffin-Manson model [111], strongly decreases with the progressing of corrosion. Zhang et al. [92] tested naturally and artificially corroded specimens and found that as corrosion progresses, a severe reduction of fatigue life can be found in steel bars. Kashani et al. [112] proposed one hysteretic model for corroded and uncorroded bars including reinforcement buckling, deterioration in post-buckling compressive strength and low-cycle fatigue effects. The calibration and validation of the hysteretic model was carried out employing an experimental data set. Buckling was simulated using the Dhakal and Maekawa model [113][114], which defines the compressive response as a function of the dimensionless slenderness ratio  $\lambda$ , defined as per Equation (2.31):

$$(2.31) \quad \lambda = \sqrt{\frac{\sigma_y}{100}} \cdot \frac{L_{eff}}{D}$$

In Equation (2.31),  $\sigma_y$  is the steel yield strength [MPa],  $L_{eff}$  is the bar buckling length and  $D$  is the bar diameter. Buckling effects are relevant when  $L_{eff}/D \geq 8$ , resulting in a post-buckling softening response.  $L_{eff}$  can be computed according to [115], considering the influence of the stiffness of horizontal ties on buckling of vertical reinforcement within RC elements. Degradation in buckling strength as a function of plastic strain demand in tension is included [116]. Corrosion affects the buckling behavior of steel bars, as reported in Kashani et al. [117], based on experimental results. In [117], empirical relations have been

proposed to modify the compressive response of corroded bars. The compressive yield strength in corroded bars  $\sigma_{y,c}$  can be estimated as per Equation (2.32):

$$(2.32) \quad \sigma_{y,c} = \begin{cases} \sigma_y \cdot (1 - 0.005 \cdot \psi); \text{ for } \frac{L_{eff}}{D} \leq 6 \\ \sigma_y \cdot (1 - 0.0065 \cdot \psi); \text{ for } 6 < \frac{L_{eff}}{D} \leq 8 \\ \sigma_y \cdot (1 - 0.0125 \cdot \psi); \text{ for } \frac{L_{eff}}{D} > 8 \end{cases}$$

Concerning the influence of corrosion on fatigue life, a modification of the Coffin-Manson model has been proposed [117]. The Coffin-Manson model is described by Equation (2.33):

$$(2.33) \quad \varepsilon_p = \theta_f \cdot (2N_f)^{-\alpha}$$

where  $\varepsilon_p$  is the plastic strain amplitude,  $2N_f$  is the number of half cycles to failure,  $\alpha$  and  $\theta$  are material constants. In [117] it is reported that  $\theta$  is not affected by corrosion and that  $\alpha$  should be modified according to Equation (2.34):

$$(2.34) \quad \alpha_c = \alpha \cdot (1 + 0.004 \cdot \psi)$$

It is worth noting that experimental evidence revealed that inelastic buckling and pit depth influence fatigue life [118][119].

### 2.3.3 Cover Concrete

The work of Vecchio and Collins [120]-[122] represented the foundation for predictive approaches used to estimate concrete compressive strength in the presence of corrosion-induced microcracks. According to Coronelli and Gambarova [123], the strength reduction depends on the magnitude of average transverse tensile strain which cause microcracks [120]. The reduced concrete strength  $f_{c,corr}$  can be computed as per Equation (2.35):

$$(2.35) \quad f_{c,corr}(t) = \frac{f_c}{1 + K \cdot \left( \frac{\varepsilon_1(t)}{\varepsilon_{c0}} \right)}$$

where  $K$  is a coefficient related to bar type and diameter and that can be taken as 0.1 for medium diameter ribbed bars [122],  $\varepsilon_{c0}$  is the strain at peak compressive stress and  $\varepsilon_1$  is the average tensile strain in cracked concrete.  $\varepsilon_1$  can be estimated as per Equation (2.36):

$$(2.36) \quad \varepsilon_1(t) = \frac{b_f - b_0}{b_0}$$

where  $b_f$  is the section width increased by corrosion cracking and  $b_0$  is the section width in uncorroded condition. The increase in section width can be estimated according to (2.37):



$$(2.37) \quad b_f - b_0 = n_{bars} w_{cr}(t)$$

where  $n_{bars}$  is the number of bars on the section side and  $w_{cr}(t)$  is the total crack width for a given corrosion level at the time  $t$ .  $w_{cr}(t)$  can be estimated using Equation (2.38), proposed by Molina et al. [19]:

$$(2.38) \quad w_{cr}(t) = \sum_i 2\pi(v_{rs} - 1)d_{rs,i}(t)$$

where  $v_{rs}$  is the ratio of volumetric expansion of oxides with respect to the uncorroded condition and  $d_{rs,i}(t)$  is the depth of corrosion attack on the  $i$ -th bar.

The work of Capozucca and Cerri [124] presented the development of a theoretical model to estimate the loss of compressive strength in concrete surrounding corroding bars and the validation of the model through comparison with experimental data. Lee et al. [125] studied the effect of CFRP repair on RC columns subject to accelerated corrosion, observing that the repair method allowed to avoid strength and stiffness losses and only a limited ductility reduction. Andisheh et al. [126] studied the effects of corroded steel reinforcement on concrete compressive strength and ultimate strain testing corroded and uncorroded RC columns. A simple model relating  $\psi$  with the reduction of compressive strength and ductility has been proposed elaborating experimental data. The results showed that corrosion-induced cracks cause more reduction on compressive strength than on ultimate strain, but the latter varies more nonlinearly than the former with corrosion level.

#### 2.3.4 Core Concrete

The compressive behavior of confined concrete is a function of volumetric ratio, yield strength and ultimate strain of transverse reinforcement. Transverse reinforcement is generally more vulnerable to corrosion than longitudinal bars because of the smaller concrete cover depth and of their smaller diameter. Corrosion-induced deterioration leads to a reduction of mechanical properties and diameter of transverse reinforcement, modifying the confinement level provided to the concrete core. To this date, only a limited number of studies address the effects of corrosion on confined concrete.

Ou et al. [127] used Mander's model [128] to compute the confined concrete properties in presence of corrosion to assess the long term seismic performance of RC bridges subjected to chloride attack. Vu et al. [129] studied the issue experimentally and proposed empirical equations based on the regression of test data, providing a complete stress-strain model based on [128]. Andisheh et al. [130] conducted an experimental investigation on the effects of corrosion on confined concrete properties and developed a general stress-strain model based on [128]. Corrosion modifies compressive strength and ultimate strain of confined concrete, altering the shape of the stress-strain curve and, in case of high level of corrosion, the slope of the descending branch in the curve. The authors concluded that transverse

reinforcement corrosion is the main factor affecting the stress-strain response of confined concrete. The confined concrete compressive strength  $f_c'^c$  and the ultimate strain of the concrete core  $\varepsilon_{cu}^c$  computed considering corrosion of reinforcing steel are described by Equations (2.39)-(2.40):

$$(2.39) \quad f_c'^c = f_c \left( 2.254 \sqrt{1 + \frac{7.94 f_l'^c}{f_c}} - \frac{2 f_l'^c}{f_c} - 1.254 \right)$$

$$(2.40) \quad \varepsilon_{cu}^c = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{su}^c}{f_c'^c}$$

Where  $f_c$  is the compressive strength of unconfined concrete,  $f_l'^c$  is the effective lateral confining pressure assumed as uniformly distributed over the surface of the confined concrete core,  $\rho_s$  is the volumetric transverse reinforcement ratio of confined core,  $f_{yh}$  is the yield strength of transverse reinforcement and  $\varepsilon_{su}^c$  is the ultimate strain of corroded transverse reinforcement. The analytical model was validated against experimental data obtained from tests on corroded RC columns with different levels of confinement and corrosion, showing good agreement with experimental results.

### 2.3.5 Bond between Steel and Concrete

The development of shear stress at the steel-concrete interface is responsible for the load transfer between concrete matrix and steel reinforcement that occurs when a RC member is loaded. The development of shear stress is essential to prevent excessive slipping between steel and concrete and to keep the strain levels of the two materials approximatively equal. It is therefore clear that corrosion on the reinforcing bars and corrosion-induced cracking in the cover matrix may lead to not developing adequate bond strength, with potential stress redistribution and altered element response. Over the years, the impact of corrosion on the bond-slip response of RC members has been extensively studied.

Al-Sulaimani et al. [131] carried out pullout and beam tests to study the influence of corrosion and cracking on bond behavior. Localized and uniform corrosion have been investigated, and different states of corrosion progression have been considered, from uncorroded to postcracking levels. It has been found that up to a certain level of corrosion bond strength increases due to the formation of corrosion products, but as corrosion progresses it decreases due to bar deterioration and crack opening. According to [132], where laboratory data have been used to investigate the effects of corrosion rate on cracking and bond strength loss, bond strength is significantly affected by the level of corrosion. The type of concrete appears to be an important factor. The work of Almusallam et al. [133] studied the bond behavior of RC elements in precracking, cracking and postcracking stages. The study evaluated also the influence of crack width on bond strength. For low levels of corrosion (0-4%), corresponding to the precracking stage, bond strength increases. This is attributed to the formation of corrosion products that fill in the

micro-gaps at the steel–concrete interface, enhancing mechanical interlock. For higher levels of corrosion (4–6%) a sharp reduction of bond strength is observed. Finally, for corrosion levels >6% ultimate bond strength does not vary much.

The research of Fu and Chung [134] confirmed this result, while the work of Amleh and Mirza [135] identified a rapid decrease of bond strength with corrosion level, up to 92% for a corrosion level of 17.5%. Auyeung et al. [136] found that the level of corrosion up to which bond strength increases is 1%, more severe levels are characterized by bond strength reduction. Exponential reduction of slip at failure has been observed even for low levels of corrosion. Remarkably, measurable bond strength exists even for severe corrosion levels. Lee et al. [137] conducted pullout tests on artificially corroded reinforcement embedded in concrete specimens and carried out numerical simulations based on the experimental data and developed bond stress-slip curves as a function of corrosion percentage. Soudki and Sherwoord [138] studied the opportunity of applying CFRP wrapping to improve the bond strength of corroded embedded bars. 32 pullout tests were conducted on wrapped and unwrapped specimens, revealing that unwrapped specimens were characterized by bond splitting failures, while wrapped specimens showed increased bond strength and failure by bar pullout. The works of Fang and coworkers [139][140] tackled the issue both experimentally and numerically. They found that confinement can counteract the loss of bond strength due to medium levels of corrosion. Given the complexity of the topic, Berto et al. [141] applied two different approaches to deal with the effect of corrosion on the bond behavior, one based on frictional law, and one based on a damage law. Comparing the numerical results with experimental data, it has been found that the latter approach is able to describe the nonlinear shape of the bond stress vs. slip curves. A comprehensive model to describe the corrosion dependent cyclic bond behavior of RC members has been proposed by Kivell [142]. The model is based on experimental data collected from cyclic pull-out tests on corroded rebars and has been applied for the development of multi-spring models simulating the plastic hinge zone of corroded bridge piers. Figure 2.5 shows the effect of corrosion on the bond-slip behavior, while Figure 2.6 reports the representation of the cyclic bond-slip model proposed in [142].

While an analysis of the entire literature is not possible, the interested reader may refer to [143]–[146] for additional information.

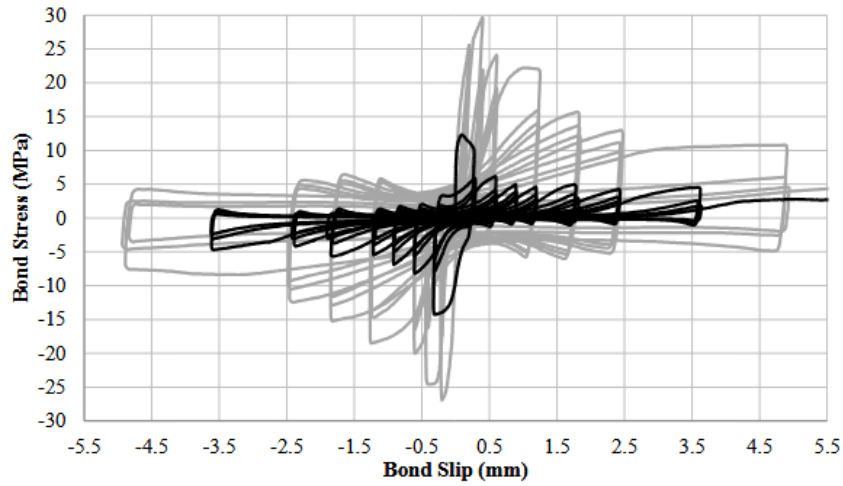


Figure 2.5. Uncorroded (grey) and corroded (black) cyclic bond-slip relationship, from Kivell [142].

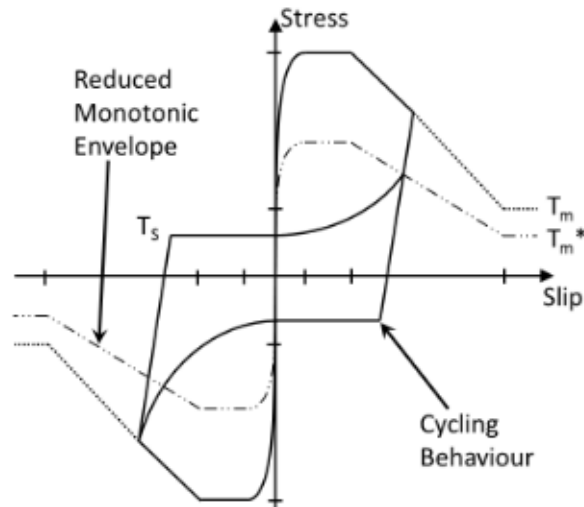


Figure 2.6. Cyclic behavior of the bond-slip model including the effects of corrosion, from Kivell [142].

## 2.4 TIME-DEPENDENT ASSESSMENT OF BRIDGE STRUCTURES

Structures and infrastructures evolve over time due to the impact of deterioration phenomena, such as corrosion, that alter material and geometric properties of RC members and consequently lead to a different performance for different times. Literature reports an infinite number of studies addressing this issue. Dizaj et al. [147] presented a stochastic

predictive methodology using a non-homogenous Markov process relating visual inspection data to the vulnerability of corroded bridges and applied it to a case study. Ni Choine et al. [148][149] studied the impact of corrosion on the columns of integral bridges and observed a significant increase in the vulnerability due to corrosion. Ghosh and Padgett [150]-[152] studied the component degradation and its impact on the seismic response of bridges in different environmental conditions, finding a significative increase of fragility and monetary losses over time. Literature studies [153][154] compared pitting and uniform corrosion models, highlighting how the former (more realistic for chloride-induced corrosion) is associated with higher Life Cycle Cost (LCC) than the latter (realistic for carbonation-induced corrosion). Capturing time-dependent structural deterioration is fundamental to aid the management of bridge structures and a rational design oriented towards durability [155][156]. Other studies explored the combination of instrumentation data and numerical methods to estimate the seismic reliability of aging transportation networks [157][158]. Biondini and coworkers studied extensively the issue of corroding bridge structures, investigating several facets of the problem, including the lifetime seismic performance of bridges exposed to corrosion [159]-[162], seismic resilience of deteriorating infrastructures [163]-[165] and modeling of the corrosion process [166]-[170].

Frangopol et al. [171][172] proposed a lifetime optimization methodology for planning inspection and repair of deteriorating structures based on reliability and LCC, oriented towards cost saving and efficiency improvement. Enright and Frangopol [173] studied the reliability of RC bridge girders through a Monte Carlo simulation approach, highlighting that the results obtained with this kind of approach are useful to predict the service life of deteriorating bridges and can be integrated into reliability-based maintenance strategies for bridges (as discussed a few years later by the same authors [174]). Lifetime reliability-based performance analysis and maintenance optimization have been developed significantly over the past 20 years [175]-[177]. Significant research efforts have been dedicated to investigating the response of bridge structures under multiple hazard attacks, such as chloride and earthquake [178]-[182], showing how long-term material deterioration has a significant impact on the seismic response of bridge structures (which evolve over time) and how a reliability and risk based approach is necessary to study failures due to multiple hazards, both on single structure and network level.

Among the available approaches to model corroded RC structures, it is possible to use a nonlinear method based on multi-layered shell elements, as proposed in Franceschini et al. [183]. The authors developed the so called “PARC\_CL 2.1” model, which is a fixed crack model implemented through a custom subroutine incorporating cyclic constitutive laws of materials, corrosion evolution over time and effects of environmental exposure. The model has been validated through comparison with experimental data. Belletti et al. [184] applied the approach to evaluate corrosion effect on PC beams without shear reinforcement,

finding that considering the actual spatial distribution of corrosion is necessary to predict the failure position. Note that modeling corrosion on prestressing strands requires ad-hoc models [185]. Vecchi and Belletti [186] simulated RC columns using different constitutive models for steel, highlighting that constitutive laws including reinforcement buckling are necessary to avoid overestimation of strength, energy dissipation, and ultimate capacity of the element, with and without corrosion, and that steel models that neglect it are not able to capture ductility and resistance deterioration of the elements. While, given the amount of available literature, it is impossible to summarize all the findings of studies on the time-dependent assessment of structures, the interested reader may refer also to [187]-[197].

## **2.5 RESEARCH GAPS**

The systematic analysis of the literature on corrosion-induced deterioration and time-dependent assessment of structures allowed to identify significant research gaps needing further research. The gaps can be summarized as:

- More accurate characterization of the parameters involved in corrosion initiation, i.e. those included in chloride diffusion and carbonation depth models;
- Robust pitting factors characterization, including the effects of material and environmental parameters;
- Robust estimation of initial and time-dependent values of corrosion rate;
- Evolution of the current models to directly include the influence of cracks on corrosion initiation;
- Correlation between corrosion rate and crack width;
- Reliable material deterioration models;
- Robust modeling approaches of corroded RC members for Life Cycle applications;
- Modern and rational integration of the PBEE principles to communicate efficiently the relevance of deterioration of RC structures to stakeholders and owners;
- Simplified approaches oriented towards reduction of the inherent computational effort of Life Cycle analysis.

It is worth stressing out that research efforts aiming at filling or reducing the gaps listed above will be fundamental in future in order to increase reliability and effectiveness of predictive analysis tools for RC structures and infrastructures.

## **2.6 CONCLUSIONS**

The different facets of corrosion-induced deterioration and time-dependent assessment have been reviewed through the systematic analysis of a significant number of research works carried out in the past. The conventional idealization of corrosion evolution has been discussed. The difference between carbonation- and chloride-induced corrosion has

been presented and the relevant models for describing chloride diffusion and carbonation depth have been reviewed. The studies related to corrosion propagation phase have been thoroughly reviewed, identifying significant uncertainty in the characterization of corrosion rate and corrosion-induced concrete cracking. Uniform and pitting corrosion models and the respective applicability conditions have been discussed. Studies related to the effects of concrete cracking on corrosion initiation and propagation have been summarized, identifying consensus only on the former. The extensive research literature on concrete, steel and bond-slip properties has been summarized. Finally, a review of significant studies on time-dependent assessment of bridge structures and components is provided. The systematic analysis of the literature allowed us to identify relevant research gaps. It is expected that research efforts oriented towards filling the identified gaps will allow to increase reliability and effectiveness of durability related predictions.

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### **3. NUMERICAL AND EXPERIMENTAL ASSESSMENT OF THE SEISMIC VULNERABILITY OF AN EXISTING BRIDGE IN PRESENCE OF CARBONATION-INDUCED CORROSION**

*This chapter is extensively based on the manuscript Reale, S., Furinghetti, M., & Pavese, A. (2026). "Numerical and experimental assessment of the seismic vulnerability of an existing bridge in presence of carbonation-induced corrosion." Soil Dynamics and Earthquake Engineering, 200, 109877.*

#### **3.1 INTRODUCTION**

Most of the Italian existing bridges have been built during the second half of XX century and are close to or have already exceeded their intended design life, with possible presence of important deterioration phenomena [1]. For this reason, existing bridges need strengthening techniques oriented to reducing the collapse risk of the structures [2]. Bridge collapses like the one of Polcevera Viaduct [3] in 2018 highlighted the impact of deterioration and made clear that assessing and managing existing bridges, considering also specific maintenance plans, is fundamental to prevent catastrophic collapses [4]-[6]. Following the event, the Italian Ministry of Infrastructure and Transportation released specific guidelines for the assessment of existing bridges [7]. A refined assessment of existing bridges is necessary because of the role these structures play in the transportation network of a nation and in post-earthquake rescue activities. In this sense, the seismic losses of a specific bridge structure can be divided into direct and indirect. The former are sustained by bridge owners and are related to direct damage on the structure, while the latter are associated with the downtime of the bridge and impact the regional economy [8].

One of the main deterioration phenomena affecting Reinforced Concrete (RC) bridges and their critical components (such as Gerber saddles) is corrosion [9]. Corrosion can be induced by carbonation or by chloride attack, depending on the specific environmental exposure. In general, corrosion damage is a main concern in bridge structures because they are directly exposed to the environment. The effects of corrosion on RC bridges have been extensively investigated in the literature. Choe et al. [10] developed probabilistic capacity models for RC columns and applied them for the fragility assessment of a corroding column. Given the seminal nature of the study, only reinforcement area loss has been considered to model corrosion effects. The authors also carried out a sensitivity analysis,

finding that the sensitivity to corrosion parameters increases in the initial years, up to the beginning of the corrosion propagation phase. An application to a single-bent highway overpass representative of highway bridges built in California has been proposed in [11], demonstrating the increase of seismic vulnerability over time due to corrosion. Ghosh and Padgett [12] proposed a full probabilistic fragility analysis methodology for multi-span continuous highway bridges investigating the impact of lifetime exposure to chlorides from deicing salts. The methodology was demonstrated through application on a multi-span continuous steel girder bridge representative of Central and Southeastern United States. Corrosion effects on RC columns have been modeled as reduction of the reinforcement area. Emphasis has also been placed on corrosion of keeper plates and anchor bolts in bearing assemblies, and increased coefficient of friction for steel bearings. The results revealed a significant increase of seismic vulnerability over time, as demonstrated by median fragility values reduction up to 27% and indicated the need for future developments including additional structural typologies and deterioration mechanisms. Given that corrosion could affect significantly the level of seismic losses on bridge structures, Ghosh and Padgett [13] proposed a component-level loss estimation methodology tailored for deteriorating bridges, based on the approach developed in [12]. The methodology accounts for time-dependent seismic vulnerability, uncertainty in repair phase and correlation between component damage. The methodology has been applied to two bridge structures representative of the Tennessee region. It has been found that including deterioration in the life cycle cost estimation leads to an increase up to 44% of the value with respect to the case neglecting corrosion. The effects of corrosion can be significant both in terms of serviceability and ultimate limit states [14]. Corrosion damage arises over time, affecting bridge functionality and resilience in the presence of extreme events [15]. Quantifying the resilience of a bridge is an important aid in the management process of an infrastructure [16]. The primary effects of corrosion on bridge elements are reinforcement area reduction, loss of concrete strength and changes in mechanical properties of reinforcing steel. Other deterioration phenomena induced by corrosion are loss of bond, reduced buckling strength, altered post-buckling behavior and low-cycle fatigue degradation [17]. It is also worth remarking that the interaction between long-term damage and extreme events has to be considered when assessing risk and reliability of bridges and networks [18][19].

This paper presents part of the research activities carried out by the authors within the framework of the collaborative research project DPC-ReLUIs 2022-24 WP3 on seismic risk and reliability of existing bridges. The study presents a case study bridge representative of existing Italian bridges built during the second half of XX century including the effects of carbonation-induced corrosion. The case study bridge is ideally placed at three reference sites in Italy. The three sites considered in the investigation are L'Aquila, characterized by high seismic hazard, Naples, characterized by moderate seismic hazard, and Milan, with low seismic hazard. For each site, a representative soil profile has been defined for foundation design, Site Response Analysis (SRA) and Soil-Structure Interaction (SSI)

modeling. For the site of L'Aquila, the simplified seismic provisions effective at the original design time required a different design of the bridge piers according to the professional practice of the period. State-of-the-art numerical models are developed and validated considering experimental results gathered during an extensive experimental campaign conducted on the case study. The impact of corrosion on the performance of a typical bridge configuration in the Italian context is quantified analyzing the structural response and the seismic reliability of the case study. With respect to the seminal studies reported in References [10]-[13], the present study is specific to the Italian context, investigating a common bridge configuration built in the second part of XX century and extending the inventory of structural configurations assessed including carbonation-induced corrosion. The corrosion damage modeling aspect has also been extended, including ad-hoc deterioration models for steel and concrete mechanical properties and validation through comparison with experimental data.

### **3.2 CASE STUDY DESCRIPTION**

The case study considered in this chapter is a railway overpass built at the end of the 60's in Northern Italy (Bridge 1 in Table 1.1 and Table 1.2). Three reference Italian sites, Milan, Naples and L'Aquila, have been assumed as locations of the structure. The structural configuration, consisting of simply supported spans on neoprene pad bearings, was common at the time and can be considered representative of existing Italian bridges built in the second part of XX century. The total length of the structure is 43.8 m, with a 19 m long central span and 12.4 m long lateral spans. The deck is composed of 14 I-shaped PC beams completed by a 0.25 m thick RC slab. The height of PC beams is 1.3 m in the central span and 0.9 m in the lateral spans. Along the deck, transverse RC beams are used to stiffen the deck section. The width of the deck is 16.5 m, with two carriageways separated by a traffic divider. Between deck and substructure, in correspondence of piers and abutments, unbolted neoprene pad bearings are placed. Each device is 0.03 m thick and has a plan section of 0.35x0.40 m. Each multicolumn bent is composed of four RC columns and a cap beam with L-shaped cross section. The columns have a square cross section with 0.8 m sides. Since the case study should be moved from its original site to three ideal sites of L'Aquila, Naples and Milan (consistently with a previous project [20]), the original design had to be checked in order to identify the possible necessity of redesigning the bridge components [21].

The original design of the bridge structure did not include seismic actions, because the original location was not included in the Italian Seismic Zones at the time. The design codes followed were the R.D. 16/11/1939, n.2229 "Norme per l'esecuzione delle opere in conglomerato cementizio semplice od armato" ("Rules for the execution of works in plain or Reinforced Concrete", translated from Italian) and the Circ. 1472 23/5/1957. Concerning the bridge design code, in the period from early 60's to late 70's the reference

was the Circ. 14/2/1962 “Norme relative ai carichi per il calcolo dei ponti stradali” (“Standard relating to loads for the calculation of road bridges”, translated from Italian), that introduced several traffic load configurations, braking loads and two conditions for wind loads (namely, the so-called “loaded structure” and “unloaded structure”).

A simulated design process has been carried out for the three ideal sites by applying the two codes mentioned above and, only for L'Aquila, also Law 25/11/1962 n.1684 “Provvedimenti per l'edilizia, con particolari prescrizioni per le zone sismiche” (“Construction measures, with particular requirements for seismic regions”, translated from Italian). The code prescribes to include seismic actions in the design as equivalent static forces. Considering seismic actions in the design leads to a slightly different design of the bent columns for the site of L'Aquila, while the original design has been confirmed by the simulated design process for the sites of Milan and Naples. The original reinforcement layout, used for the sites of Milan and Naples, and the layout obtained through the simulated design process including seismic actions, used for the site of L'Aquila, are reported in Figure 3.1. Figure 3.2 displays a sketch of the case study bridge. The concrete strength  $f_c$  was equal to 26 MPa. For the reinforcement, AQ50 class steel was used and according to the properties of steel in the reference time period [22] a yield strength  $f_y$  of 371.9 MPa has been assumed.

Similar considerations apply to the foundations of the case study. Given that the soil profiles associated with L'Aquila, Milan and Naples are different from that of the original site, foundations have been redesigned according to the professional practice of the time. Pile foundations have been designed for piers and abutments. The soil profiles for the three ideal sites are the same considered in [20]. The profiles have been defined by attempting to get a realistic representation of the geological characteristics of the three sites and are based on the experimental data provided in [23]-[27]. Table 3.1 reports the mechanical properties of the subsoil used for foundation design, SRA and SSI. The subsoil has been characterized in terms of depth of the lower boundary of the layers  $z_b$ , soil unit weight  $\gamma$ , cohesion  $c$ , angle of shearing resistance  $\varphi$ , undrained shear strength  $S_u$  and average value of small-strain shear wave velocity  $V_{s,m}$ . The pile foundation configurations are reported in Table 3.2. The piles have a diameter of 1.2 m and length  $L_{pil}$ . Foundation configurations are described in terms of rows of foundation piles parallel ( $n_{pl}$ ) and perpendicular ( $n_{pt}$ ) to the bridge longitudinal axis.

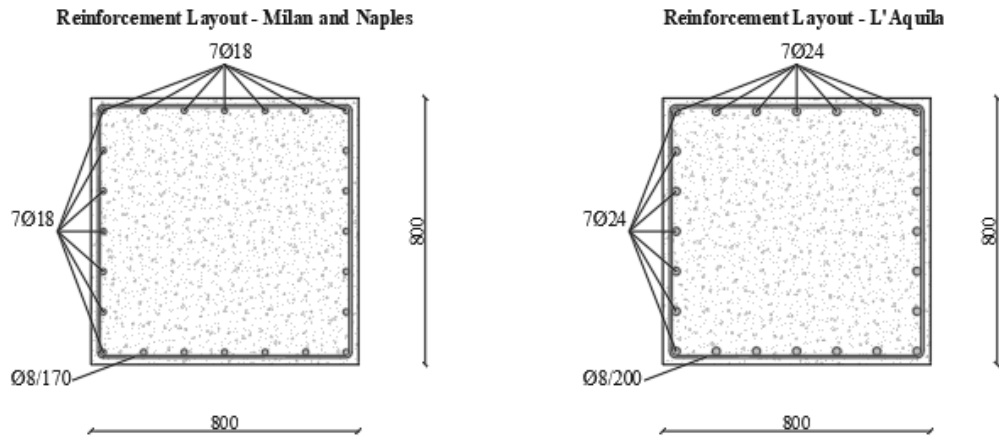


Figure 3.1. Reinforcement layout for the bent columns, for the sites of Milan, Naples and L'Aquila.

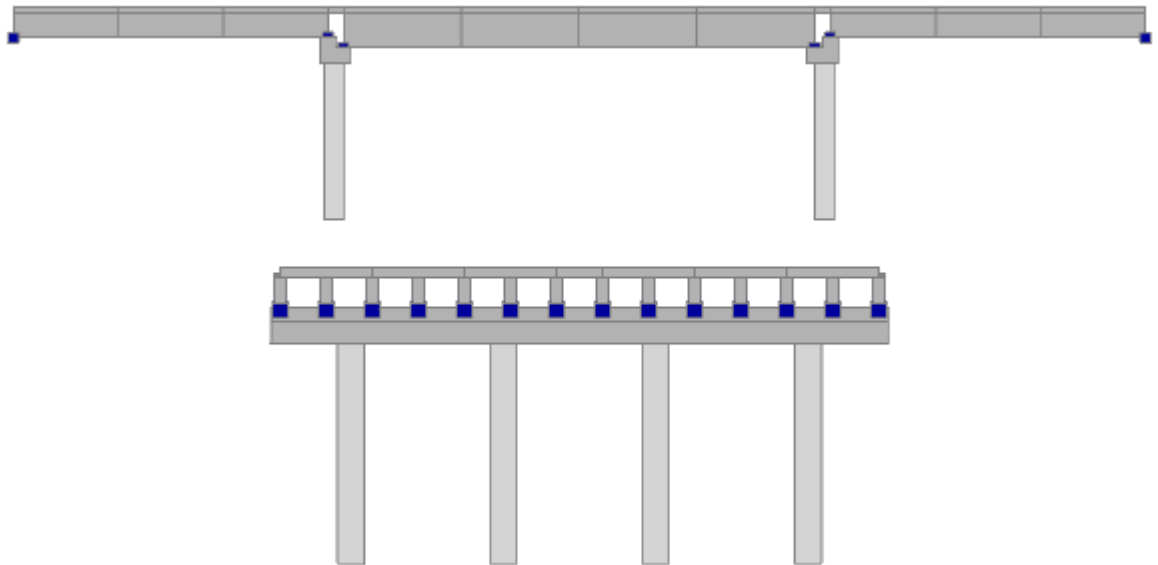


Figure 3.2. Sketch of the case study bridge configuration.

Table 3.1. Mechanical properties of the subsoil in the three ideal sites, adapted from [20].

| Site     | Layer | Description          | $z_b$<br>[m] | $\gamma$<br>[kN/m <sup>3</sup> ] | $K_0$ | $c'$<br>[kPa] | $\varphi'$<br>[°] | $S_u$<br>[kPa] | $V_{s,m}$<br>[m/s] |
|----------|-------|----------------------|--------------|----------------------------------|-------|---------------|-------------------|----------------|--------------------|
| L'Aquila | 1     | Silt                 | 3.0          | 20.0                             | 0.56  | 10.0          | 26                | 75             | 180                |
|          | 2     | Silt                 | 10.0         | 20.0                             | 0.59  | 10.0          | 24                | 150            | 256                |
|          | 3     | Clay                 | 18.5         | 20.0                             | 0.59  | 10.0          | 24                | 300            | 295                |
|          | 4     | Stiff clay           | 95.0         | 20.0                             | 0.59  | 10.0          | 24                | 300            | 825                |
|          | 5     | Bedrock              | -            | 22.0                             | -     | -             | -                 | -              | 1000               |
| Naples   | 1     | Sand and debris      | 5.0          | 20.0                             | 0.41  | 0.0           | 36                | -              | 360                |
|          | 2     | Silty sand           | 30.4         | 18.5                             | 0.41  | 0.0           | 36                | -              | 389                |
|          | 3     | Yellow tuff          | 47.4         | 17.0                             | -     | 1750.0        | 0                 | -              | 704                |
|          | 4     | Green tuff (Bedrock) | -            | 22.0                             | -     |               |                   | -              | 790                |
| Milan    | 1     | Gravelly sand        | 26.0         | 20.0                             | 0.46  | 0.0           | 33                | -              | 252                |
|          | 2     | Clay                 | 70.0         | 20.0                             | 0.43  | 0.0           | 35                | -              | 400                |
|          | 3     | Bedrock              | -            | 22.0                             | -     | -             | -                 | -              | 1000               |

Table 3.2. Foundation configurations for the three ideal sites.

|            | L'Aquila |          |           | Naples   |          |           | Milan    |          |           |
|------------|----------|----------|-----------|----------|----------|-----------|----------|----------|-----------|
| Position   | $n_{pl}$ | $n_{pt}$ | $L_{pil}$ | $n_{pl}$ | $n_{pt}$ | $L_{pil}$ | $n_{pl}$ | $n_{pt}$ | $L_{pil}$ |
| Abutment 1 | 4        | 6        | 35        | 4        | 6        | 14        | 4        | 6        | 16        |
| Pier 1     | 2        | 6        | 25        | 2        | 6        | 11        | 2        | 6        | 14        |
| Pier 2     | 2        | 6        | 25        | 2        | 6        | 11        | 2        | 6        | 14        |
| Abutment 2 | 4        | 6        | 35        | 4        | 6        | 14        | 4        | 6        | 16        |

### 3.3 STRUCTURAL MODELING

#### 3.3.1 Bridge Superstructures

The tri-dimensional numerical model of the case study has been developed in OpenSees [28]. This section provides details on the modeling of the different components of the bridge, illustrating the development of the model (illustrated in Figure 3.3).

The superstructure, composed of I-shaped PC beams and RC slab, is modeled using a refined grillage model. According to the common practice, the superstructure is assumed to behave linearly during seismic events. Longitudinal girders and transverse stiffeners have been modeled using elastic beam elements. The elastic properties of PC elements have been considered in the model [29]. The RC slab has been modeled using *ShellMITC4* elements [30]. Rigid elements have been used to connect the slab with the beams, as displayed in Figure 3.3. The grillage model of the deck has been used in order to describe in a realistic way the variation of axial load on the bearings.

Neoprene pad bearings have been modeled employing *flatSliderBearing* elements [28]. The element has coupled friction properties for the shear deformations and is able to capture the uplift behavior. Neoprene pads with small rubber thickness and low load pressure are usually associated with a slipping failure mode [31], which can be idealized as linear viscoelastic behavior followed by pure friction behavior. In presence of seismic events, the devices have often been identified as the most critical components of bridge structures [32][33]. The linear viscoelastic behavior is characterized by the shear stiffness  $K$ , which can be computed as in Equation (3.1):

$$(3.1) \quad K = \frac{GA}{t_r}$$

where  $G$  is the rubber shear modulus assumed as equal to 1 MPa,  $A$  is the cross section area of rubber and  $t_r$  is the total thickness of the rubber layers. The value of the neoprene-concrete friction coefficient  $\mu$  is affected by a significant variability in the scientific literature [34]. In this work, the formulation proposed by Schrage [35] has been considered. The relationship between  $\mu$  and the vertical compressive stress  $\sigma_v$  (expressed in N/mm<sup>2</sup>) is defined as per Equation (3.2):

$$(3.2) \quad \mu = 0.05 + \frac{0.4}{\sigma_v}$$

For the present case study, the value of  $\mu$  computed considering the average vertical stress on the devices was equal to 0.39. Note that this value is aligned with that provided by Caltrans Guidelines [36], that indicate a value of 0.4. Given the lack of data regarding the velocity-dependent behavior of unbolted neoprene pad bearings and that, according to the results reported in [34] for this type of bearing devices,  $\mu$  values close to 0.4 are associated with more conservative estimates of seismic risk and losses, a Coulomb friction model with  $\mu$  equal to 0.39 has been used. This assumption is consistent with [34] and could be refined in future developments of the present study. The conceptual response of the devices is represented in Figure 3.4.

The bent caps have been modeled using elastic beams, considering their cracked properties [29]. To avoid localization problems, bent columns have been modeled with distributed plasticity elements with force-based formulation employing a Gauss-Radau integration method [37]. Six integrations points are used in the elements. The plastic hinge length  $L_p$  was calculated according to Priestley et al. [29]. The element central sections have been defined as elastic, with cracked properties derived from moment-curvature analysis of the column sections. The element end sections have been defined through a fiber-based approach. Steel fibers have been defined using the Giuffr -Menegotto-Pinto Model with isotropic strain hardening, based on [38] and [39]. In order to account for bar fractures, the *MinMax* material wrapper has been applied setting a strain limit of 0.1 as recommended in [29]. Concrete cover fibers have been defined using the Kent-Scott-Park model [40], while core fibers have been modeled with the Popovics model [41]. Degraded linear unloading and reloading stiffness of the concrete fibers has been described referring to the work of Karsan and Jirsa [42]. The properties of confined concrete have been defined according to the model of Mander et al. [43]. P-Delta effects have been included to consider geometrical nonlinearities in the model. The parameters used for defining the structural model, namely  $f_c$  and ultimate strain  $\varepsilon_{cu}$  for cover and core concrete,  $f_y$ , steel ultimate strength  $f_u$ , steel ultimate strain  $\varepsilon_{su}$ , steel Young modulus  $E_s$ ,  $\mu$  and  $K$  have been reported in Table 3.3. The mesh discretization adopted to model the piers has been illustrated in Figure 3.5. It can be observed that one inelastic beam (commonly referred to as *BeamwithHinges*) has been used to represent each column, modeling dissipative and non-dissipative regions within the same element. It is worth noting that, due to the lack of experimental data,  $L_p$  in corroded condition has been computed according to [29], using corroded model properties.

The interaction between deck and abutment backwall is modeled using *ZeroLength* elements and the *ElasticPPGap* material, as displayed in Figure 3.3. Material parameters have been defined according to the indications reported in [36] for seat type abutments. Material damage has been considered, avoiding recentering in presence of load reversals. Bridge system response has been preliminarily validated, assuming fixed base condition, against that of an independent model developed using the Sap2000 software [44]. Validation of the OpenSees fixed base structural model has been carried out by considering gravity load and modal analysis. For the former, the difference between the results obtained with the two independent models is around 1%. For the latter, agreement between the two models in the estimation of the first three modal shapes (translation along the bridge longitudinal axis, translation along the transverse direction and rotation around the vertical axis) has been found, with maximum difference around 1% in terms of computed modal periods.



### 3.3.2 SSI

SSI effects have been included through the uniaxial, inertial macroelements (1DME) proposed in [45][46]. The macroelements have been used to model abutments and piled foundations of the piers, as in a previous research project [20].

1DMEs are subjected to seismic input determined through SRA and applied at the effective depth of the geotechnical systems [21]. The fundamental concepts related to the 1DMEs are here recalled, but the interested reader may refer to [20][47] for details on the calibration. 1DMEs, consisting in a nonlinear spring in parallel with a linear dashpot for each relevant degree of freedom and assuming decoupled behavior between the degrees of freedom, have been used to simulate frequency- and amplitude-dependent dynamic SSI effects. Amplitude-dependent effects have been simulated through a linear elastic perfectly plastic constitutive law. The 1DMEs used for the abutments include the three translational degrees of freedom and relate the interaction force exchanged between deck and front wall with the corresponding displacement. The 1DMEs used for pier foundations include also two rotational degrees of freedom and relate force or moment exchanged between pier and foundation raft to the corresponding displacement or rotation. Frequency-dependent effects have been simulated through the combination of 1DMEs, foundation masses and modal masses of the geotechnical systems [20]. 1DMEs have been implemented through *TwoNodeLink* elements, connected in parallel to viscous dampers to reproduce radiation and viscous damping effects. It is worth noting that the connection between deck and abutment macroelements has been modeled through the arrangement described in [20][21].

**Table 3.3. Parameters used for structural model definition.**

| Parameter                                   | Value  |
|---------------------------------------------|--------|
| $f_{c,cover}$ [MPa]                         | 26.00  |
| $\varepsilon_{ch,cover}$ [-]                | 0.005  |
| $f_{c,core} - \text{MI, NA}$ [MPa]          | 26.66  |
| $\varepsilon_{ch,core} - \text{MI, NA}$ [-] | 0.007  |
| $f_{c,core} - \text{AQ}$ [MPa]              | 26.57  |
| $\varepsilon_{ch,core} - \text{AQ}$ [-]     | 0.007  |
| $f_y$ [MPa]                                 | 371.90 |
| $f_u$ [MPa]                                 | 531.00 |
| $\varepsilon_{su}$ [-]                      | 0.10   |
| $E_s$ [MPa]                                 | 200000 |
| $\mu$ [-]                                   | 0.39   |
| $K$ [kN/m]                                  | 4667   |

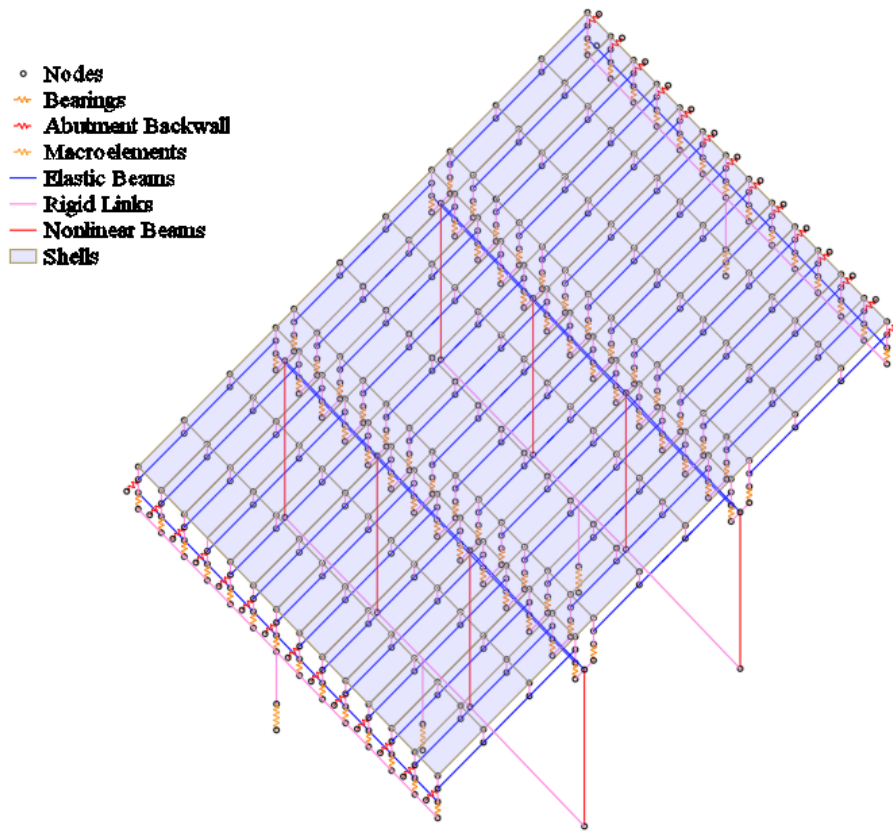


Figure 3.3. Structural Model representation.

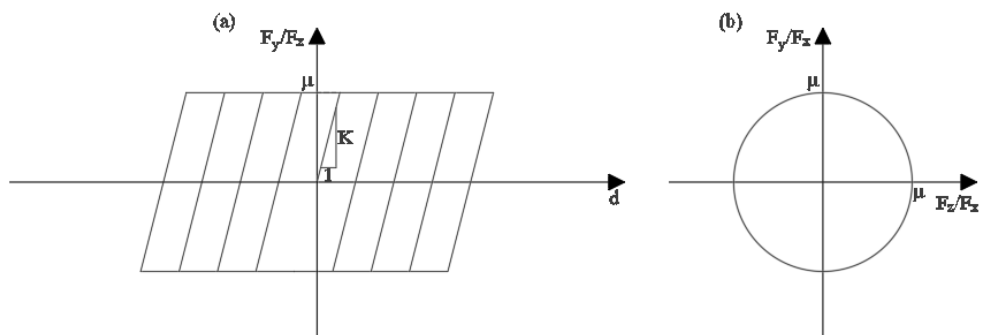


Figure 3.4. Representation of bearing devices response. (a) Cyclic response. (b) Circular yield surface in shear.

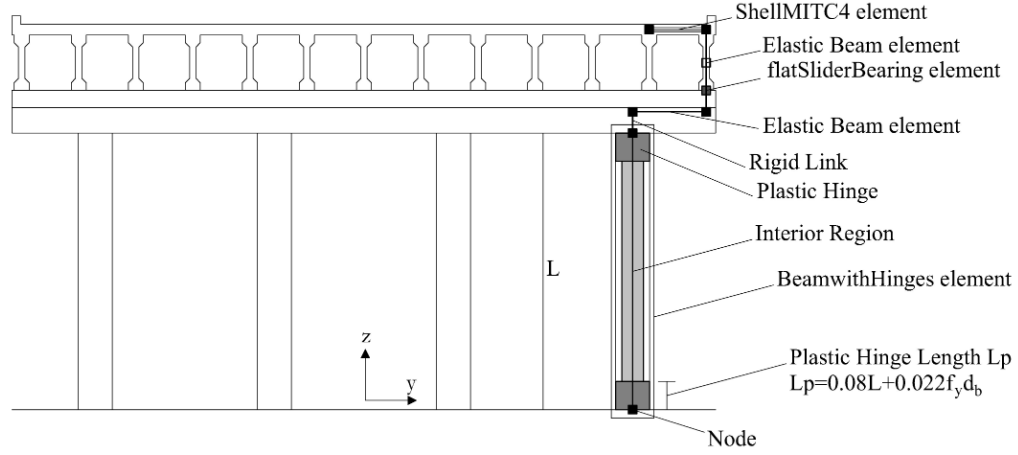


Figure 3.5. Mesh discretization adopted to model the piers.

### 3.4 CORROSION DAMAGE

#### 3.4.1 Corrosion scenario

In order to investigate the effects of carbonation-induced corrosion on the seismic response of the case study bridge, a representative corrosion scenario has been identified according to the real location of the structure. Carbonation-induced corrosion effects have been investigated since the bent columns denoted signs of carbonation and corrosion during in-site inspections.

Carbonation takes place when carbon dioxide, which is abundant in the atmosphere, penetrates the concrete volume. It then begins reacting with hydroxides (such as calcium hydroxide), resulting in the formation of carbonates. The reaction leads to a reduction of the pH value of the pore solution below the value of 8.5. The immediate consequence is the formation of an unstable passive film on the surface of the rebars [48]. The evolution of corrosion can be divided into two phases: initiation and propagation. The first phase can be described through a parabolic penetration law [49][50]:

$$(3.3) \quad s_{carb} = k_{carb} \cdot \frac{1}{t^{n_{carb}}}$$

In Equation (3.3),  $s_{carb}$  is the thickness of the carbonated layer,  $k_{carb}$  is the penetration rate coefficient,  $t$  is the time and  $n_{carb}$  is a parameter that can be taken as 2 for normally compacted concrete [50].

Corrosion starts when the carbonated depth reaches the reinforcement, destroying the passive layer. Concrete resistivity is the most important parameter in the propagation phase [51]. To define the time-dependent material properties, it is fundamental to define the time-dependent diameter loss of longitudinal and transverse reinforcement. According to the literature, a uniform corrosion model assuming that the section loss is equally distributed around the rebar can be used [52]:

$$(3.4) \quad d(t) = d_0 - 2i_{corr}0.0116(t - t_i)$$

In Equation (3.4),  $d(t)$  represents the time-dependent diameter of the reinforcement [mm],  $i_{corr}$  is the corrosion current density [ $\mu\text{A}/\text{cm}^2$ ],  $t$  is the current time [years] and  $t_i$  is the corrosion initiation time [years]. In this work, a moderate corrosion scenario has been considered. In this scenario, a representative value of  $i_{corr}$  can be  $0.8 \mu\text{A}/\text{cm}^2$  [53]. Assuming a water to cement ratio of 0.6, a  $t_i$  value of 13.5 years has been identified [54]. Finally, the section loss  $\psi$  for the  $i$ -th bar can be computed as per Equation (3.5):

$$(3.5) \quad \psi = \frac{A_0 - A(t)}{A_0} \cdot 100 [\%]$$

where  $A_0$  is the area of uncorroded reinforcement bars and  $A(t)$  is the time-dependent section of the rebars.

Considering the current time, 53 years after the construction of the case study bridge,  $\psi$  values for longitudinal and transverse reinforcement (for original and seismic design) have been computed and the results have been reported in Table 3.4 and Table 3.5. As expected, the  $\psi$  value for transverse reinforcement is higher than the corresponding value for longitudinal bars due to the smaller bar size.

Table 3.4. Section loss values at  $t=53$  years, original design.

| $d_0$<br>longitudinal<br>[mm] | $d(t)$<br>longitudinal<br>[mm] | $\psi$<br>longitudinal<br>[%] | $d_0$<br>transverse<br>[mm] | $d(t)$<br>transverse<br>[mm] | $\psi$ transverse<br>[%] |
|-------------------------------|--------------------------------|-------------------------------|-----------------------------|------------------------------|--------------------------|
| 18                            | 17.27                          | 7.98                          | 8                           | 7.27                         | 17.49                    |

Table 3.5. Section loss values at  $t=53$  years, design including seismic actions.

| $d_0$<br>longitudinal<br>[mm] | $d(t)$<br>longitudinal<br>[mm] | $\psi$<br>longitudinal<br>[%] | $d_0$<br>transverse<br>[mm] | $d(t)$<br>transverse<br>[mm] | $\psi$ transverse<br>[%] |
|-------------------------------|--------------------------------|-------------------------------|-----------------------------|------------------------------|--------------------------|
| 24                            | 23.27                          | 6.02                          | 8                           | 7.27                         | 17.49                    |

### 3.4.2 Corrosion effects on steel material properties

Steel material properties, namely  $f_y$ ,  $f_u$  and  $\varepsilon_{su}$  have been modified to consider corrosion effects.  $f_y$  and  $f_u$  have been modified according to the relations proposed by Du et al. [55][56]. The work of Biondini and Vergani [57] has been considered to modify  $\varepsilon_{su}$ . Equations (3.6)-(3.8) have been used to modify the steel material properties:

$$(3.6) \quad f_{y,c} = [1 - 0.005\psi]f_y$$

$$(3.7) \quad f_{u,c} = [1 - 0.005\psi]f_u$$

$$(3.8) \quad \varepsilon_{su,c} = \begin{cases} \varepsilon_{su} & \text{if } 0 \leq \delta_s \leq 0.016 \\ 0.1521\delta_s^{-0.4583}\varepsilon_{su} & \text{if } \delta_s > 0.016 \end{cases}$$

where  $f_{y,c}$  is the yield strength in corroded reinforcement,  $f_{u,c}$  is the ultimate strength in presence of corrosion,  $\varepsilon_{su,c}$  is the ultimate strain of corroded steel and  $\delta_s$  is the damage function proposed in [57].

### 3.4.3 Corrosion effects on concrete material properties

Cover concrete compressive strength is reduced to consider the effects of cracks caused by the expansion of corrosion products. For this purpose, the approach proposed by Coronelli and Gambarova [58], based on the work of Vecchio and Collins [59], is used. The reduced concrete cover strength  $f_{c,corr}$  is computed as per Equation (3.9):

$$(3.9) \quad f_{c,corr} = \frac{f_c}{1 + \frac{K\varepsilon_1}{\varepsilon_{c0}}}$$

Where  $K$  is a coefficient related to bar roughness and diameter and that can be taken as 0.1 for medium diameter ribbed bars,  $\varepsilon_{c0}$  is the strain at peak compressive stress and  $\varepsilon_1$  is the average tensile strain in the concrete due to the expansion of corrosion products:

$$(3.10) \quad \varepsilon_1 = \frac{b_f - b_0}{b_0}$$

In Equation (3.10),  $b_f$  is the section width increased by corrosion cracking, and  $b_0$  is the section width in pristine condition. The term  $b_f - b_0$  can be estimated using Equation (3.11):

$$(3.11) \quad b_f - b_0 = n_{bars}w_{cr}$$

Where  $n_{bars}$  is the number of bars on the side of the section and  $w_{cr}$  is the total crack width for a corrosion level  $X$ . According to the formulation of Molina et al. [60],  $w_{cr}$  can be expressed as:

$$(3.12) \quad w_{cr} = 2\pi(v_{rs} - 1)d_{rs}$$

In Equation (3.12),  $v_{rs}$  is the volumetric expansion ratio of the corrosion products and  $d_{rs}$  is the depth of the corrosion attack.  $d_{rs}$  has been estimated as per Equation (3.13):

$$(3.13) \quad d_{rs} = \frac{d_o - d(t)}{2}$$

Considering the high corrosion percentage in transverse reinforcement, the effects of confinement on core concrete strength and ductility have been neglected. Note that this assumption can be reasonable considering that due to the relation between spacing (170 mm for the original design, 200 mm for the redesigned section), number of legs (2) and size of the stirrups (8 mm diameter) and size of the RC bent columns (800x800 mm) confinement effects are moderate even in uncorroded condition.

### 3.5 SECTION AND PUSHOVER ANALYSIS

The effects of corrosion on the bent columns have been first analyzed through Moment-Curvature (Figure 3.6, considering an axial load equal to 1493.48 kN) and Pushover analysis in transverse direction (Figure 3.7), for each of the three ideal sites considered in the present study. It is worth noting that, due to the simulated design process carried out to take into account seismic actions, the reinforcement layout of the bent columns for the site of L'Aquila is slightly different from the reinforcement layout used for Milan and Naples.

The ultimate curvature corresponds to the crushing of the concrete core in both pristine and corroded condition. The section with corroded reinforcement, considering the design for Milan and Naples, is characterized by a 57% reduction of the ultimate curvature. The ultimate curvature reduction is equal to 50% for the site of L'Aquila. In terms of peak moment, the reduction is 18% for the sites of Milan and Naples and 17% for the site of L'Aquila. As displayed in Figure 3.7, the ultimate drift of the elements is associated with concrete crushing, both in pristine and corroded condition. For the sites of Milan and Naples, a 56% reduction of the ultimate drift has been observed. For the site of L'Aquila, a 47% reduction of the ultimate drift has been observed. This effect is due to the loss of confinement (and consequently of ductility) in the confined concrete core. The reduction in peak base shear is 16% for all the sites and is mainly related to the reduction of reinforcement area. The pushover analysis also reveals that the stiffness of the bents with corroded columns is lower than that of the pristine elements. This can be explained

considering the reduction in reinforcement area and material strength of the columns, which directly influence the stiffness of the elements.

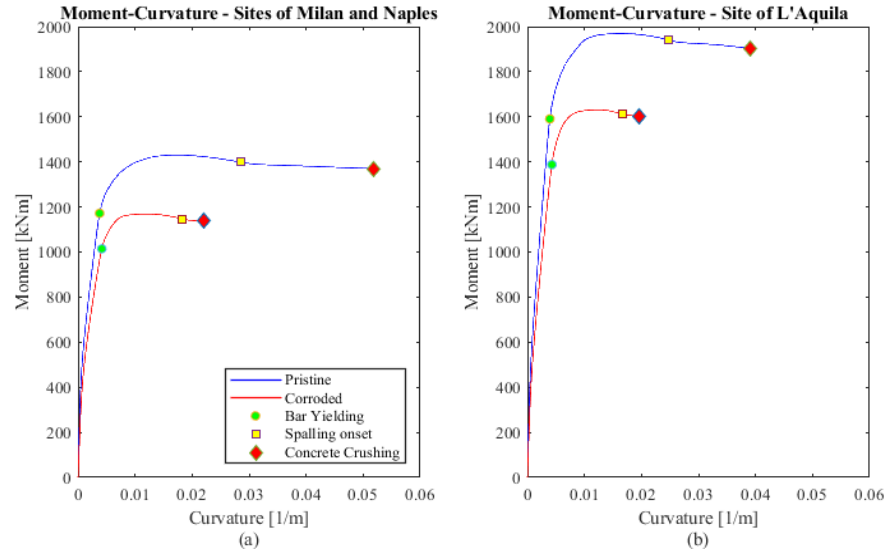


Figure 3.6. Time-dependent Moment-Curvature relations for the bent columns. (a) Sites of Milan and Naples. (b) Site of L'Aquila.

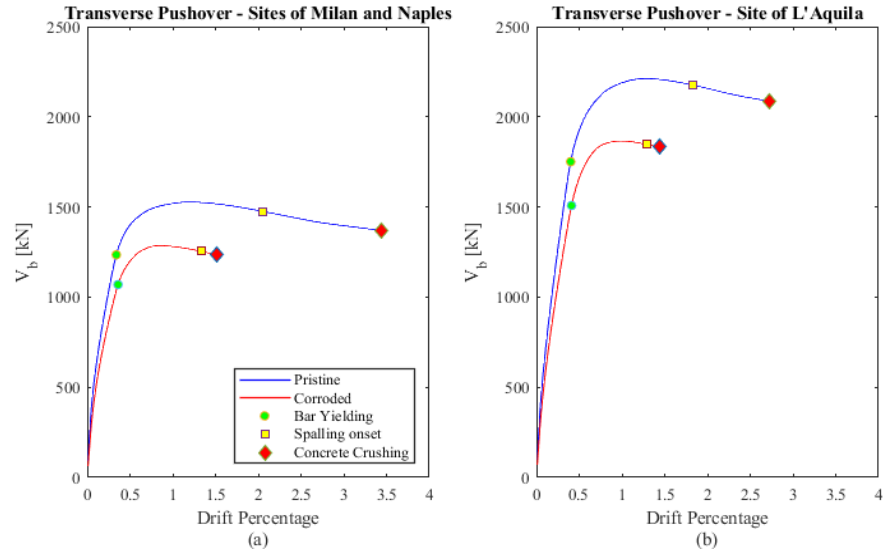


Figure 3.7. Time-dependent pushover curves of the piers in transverse direction. (a) Sites of Milan and Naples. (b) Site of L'Aquila.

### 3.6 VALIDATION OF THE CORRODED MODEL

#### 3.6.1 Experimental Campaign

The case study bridge was the subject of an extensive experimental campaign to characterize the material properties and the damage distribution. The experimental campaign consisted of destructive and non-destructive tests to characterize steel and concrete properties. Table 3.6 reports a summary of the experimental campaign. The destructive tests have been conducted at the laboratory of University of Pavia.

The concrete compressive tests revealed that the average carbonation penetration depth was around 80 mm. The mean concrete strength obtained through the compressive test, considering specimens taken from several positions, was 12.99 MPa. The values of  $f_y$  and  $f_u$  obtained from the tensile tests were 360 MPa and 527 MPa respectively. The results of the non-destructive tests confirmed those of the destructive tests for the highly deteriorated regions and the assumptions made for the modeling of the pristine condition bridge.

A detailed laser-scanner survey has been conducted to characterize the damage to bridge components. Severe deterioration has been identified in the bent columns, with spalling in some sections. The carbonation penetration depth identified using phenolphthalein suggests the possible corrosion of reinforcement along the length of the columns. Figure 3.8 displays the geometry of one of the bents obtained from the survey, while Figure 3.9 shows the damaged sections of a bent column. An example of the damage detected in the columns is reported in Figure 3.10.

**Table 3.6. Summary of the experimental campaign.**

| Number of tests | Test Type                 |
|-----------------|---------------------------|
| 8               | Concrete Compressive Test |
| 2               | Steel Tensile Test        |
| 8               | Schmidt Hammer Test       |
| 4               | Concrete Ultrasonic Test  |
| 24              | Pull-out test             |
| 1               | Covermeter investigation  |



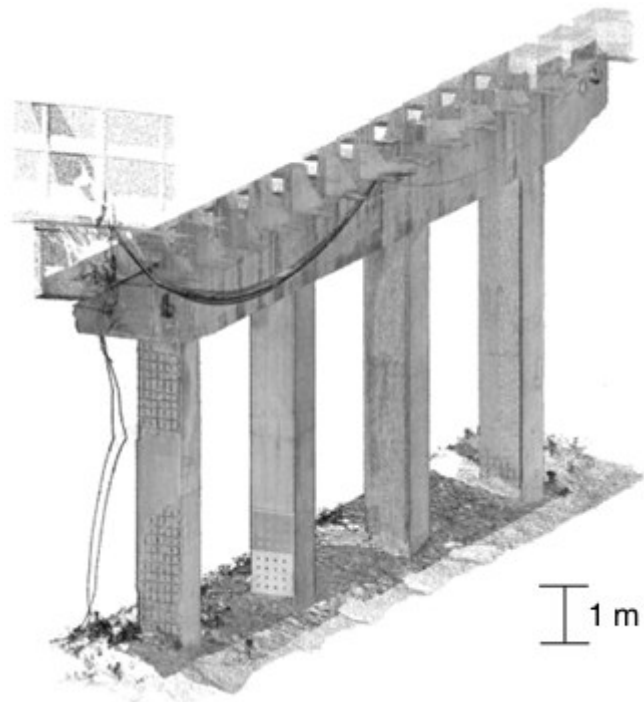


Figure 3.8. Laser-scanner survey of one bent with visible damage.

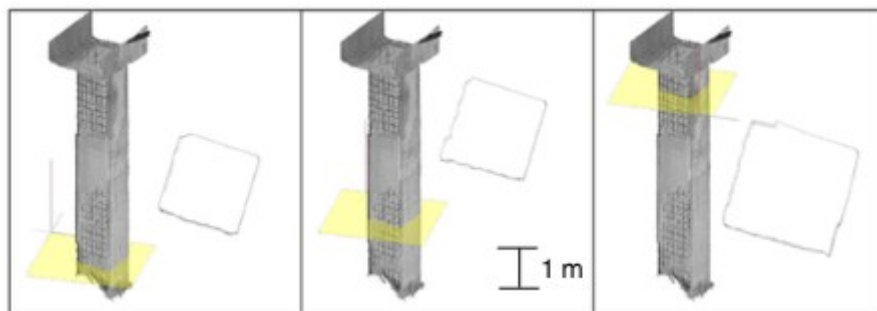


Figure 3.9. Damaged sections of a column.



Figure 3.10. Damage on bent column with spalling of cover concrete and exposed reinforcement.

### 3.6.2 Validation of the corroded bridge scenario

The experimental data collected from the experimental campaign have been used to validate the corrosion scenario assumed for the case study. The values of  $d(t)$ ,  $f_{c,c}$ ,  $f_{y,c}$  and  $f_{u,c}$  computed with the corrosion damage models previously introduced for the original design of the columns have been compared with the experimental results to assess the validity of the assumptions made to define the corrosion scenario. For each parameter, the numerical and experimental values have been compared and the relative error in the numerical data has been computed. The results are reported in Table 3.7. As can be observed, the correspondence between the numerical results obtained for the assumed corrosion scenario and the experimental data collected in the campaign is very strong, with almost negligible errors. The comparison confirms the validity of the corrosion scenario assumed for the case study, providing a strong foundation for the results obtained in the following sections.

Table 3.7. Comparison between numerical and experimental data.

| Parameter           | Numerical | Experimental | Error [%] |
|---------------------|-----------|--------------|-----------|
| $f_{c,c}$           | 12.95     | 12.99        | -0.31     |
| $f_{y,c}$           | 357.06    | 360.00       | -0.82     |
| $f_{u,c}$           | 524.88    | 527.00       | -0.40     |
| $d(t)$ longitudinal | 17.27     | 17.87        | -3.36     |

### 3.7 STRUCTURAL RESPONSE

The structural response has been assessed through NTLHAs with tri-component ground motions. Multiple Stripe Analysis (MSA) [61] has been performed for the case study. 10 Intensity Measure (IM) levels have been considered. The IM levels can be grouped as low (IM levels 1-3), medium (IM levels 4-7) and high (IM levels 8-10) seismic intensities. For each IM level, a suite of 20 ground motions triplets has been employed to analyze the bridge. Ground motions have been selected from Soil Category A [62] records and have been subjected to SRA considering specific foundation depths and soil profiles [21].

#### 3.7.1 Modal Analysis

In order to define the conditioning period for the selection of the input motion, Modal Analysis has been performed for the case study bridge, for each site, in pristine and corroded condition (the periods corresponding to the first three modal shapes are reported in Table 3.8) considering the linearized tangent stiffness matrix assembled from the uniaxial material tangents at equilibrium state, using a general eigensolver. According to the results of the analysis, a conditioning period  $T_c$  of 0.5 s has been identified. It can be observed that the fundamental periods are slightly different between each site, considering the same condition. This is due to the fact that the macroelements used for each site have been defined separately and that the reinforcement layout in the bent columns, which has a minor influence on the stiffness of the pier, is different between L'Aquila and the other sites. Considering pristine and corroded conditions, a slight lengthening of the fundamental periods has been observed. This result was expected and can be explained considering that the amount of reinforcement is lower in the corroded models and that the mechanical properties of steel and concrete have been reduced with respect to the pristine condition due to the presence of corrosion. It is worth noting that the numerical results are consistent with the experimental data reported in [63], indicating negligible differences in the eigenfrequencies due to corrosion effects in no-damage condition, and with the numerical investigation carried out in [52], highlighting a slight increase of the fundamental periods because of the reduction of pier stiffness. Considering the modal shapes of the three fundamental modes, which are associated with almost the entire participating mass for this structural configuration, no significant variation occurred due to corrosion effects. This result has also been observed in [52]. The first modal shape is longitudinal, the second is transversal and the third is rotational, both in pristine and corroded condition, as depicted in Figure 3.11.

Table 3.8. Modal Analysis of the case study bridge for each site, in pristine and corroded condition.

| Site | Condition |          | Site | Condition |          | Site | Condition |          |
|------|-----------|----------|------|-----------|----------|------|-----------|----------|
| AQ   | Pristine  | Corroded | NA   | Pristine  | Corroded | MI   | Pristine  | Corroded |
| n    | T [s]     | T [s]    | n    | T [s]     | T [s]    | n    | T [s]     | T [s]    |
| 1    | 0.517     | 0.530    | 1    | 0.531     | 0.547    | 1    | 0.532     | 0.548    |
| 2    | 0.508     | 0.525    | 2    | 0.525     | 0.545    | 2    | 0.526     | 0.546    |
| 3    | 0.387     | 0.391    | 3    | 0.391     | 0.395    | 3    | 0.391     | 0.395    |

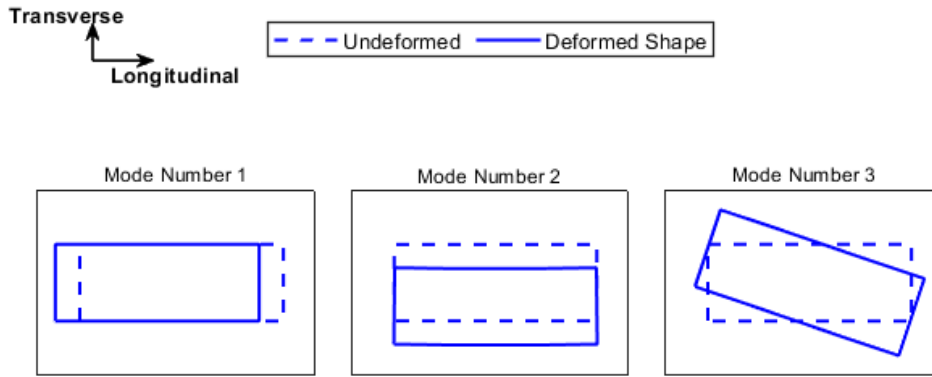


Figure 3.11. Modal shapes for the three fundamental modes of the case study bridge.

### 3.7.2 Seismic Hazard, Ground Motion Selection and SRA

Site specific Probabilistic Seismic Hazard Analysis (PSHA) has been carried out for the three ideal sites considered in the investigation in the framework of the RINTC project [64], consistently with the current Italian Building Code. Seismic Hazard Curves for 5% damped spectral accelerations  $S_a(T_i)$  on Category A soil [62] at regularly spaced vibration periods  $T_i$  have been evaluated using the seismic source model provided by Meletti et al. [65], magnitude distribution and rates from Barani et al. [66] and the Ground Motion Prediction Equation (GMPE) of Ambraseys et al. [67] (for  $T_i$  up to 2 s) and Akkar and Bommer [68] (for  $T_i$  beyond 2 s).

Input ground motion selection has been carried out according to the approach proposed in [69], based on the Conditional Spectrum (CS) method [70]. The record selection method aims at maintaining a higher order of hazard consistency indirectly accounting for metrics depending on magnitude  $M$  and closest to rupture distance  $R$ , such as duration, but not captured in the response spectra. The main steps of the approach are the following [20][69]:

- Perform seismic hazard deaggregation [71] conditional on  $S_a(T_i)$  for each of the 10 IM level corresponding to a return period  $T_R$  equal to 10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 years;
- Define the CS for each IM level, using one of the previously introduced GMPEs [67][68];
- Sample 20 response spectra for the CS distribution for each IM level;
- Select 20 real ground motions matching the simulated spectra for each IM level;
- For each IM level, post process the selected ground motion set consistently with the deaggregation results.

For each IM level, 20 ground motion triplets have been selected. Compatibility with the target distribution has been checked considering the horizontal components. Ground motions have been selected from Soil Category A [62] because of the SRA conducted prior to performing MSA. Consistently with literature [72][73],  $S_a(T_i)$  has been used as IM.

SRA has been performed for each of the three sites according to the subsoil profiles described in Table 3.1 to compute seismic motion at effective depth. The ground water table is located at a depth of 2 m from ground level. 1D subsoil models have been defined consistently with the information reported in Table 3.1. Equivalent viscoelastic analysis in the frequency domain was carried out assuming that the horizontal motion was produced by shear waves and the vertical one by compression waves. Depending on the type of soil, the dependence of shear modulus and damping ratio on the strain amplitude has been simulated through the relationships reported in [74]-[76]. The interested reader may refer to [20] for a comprehensive discussion on the SRA. Figure 3.12 presents the 5% damped response spectra of the input ground motions for IM levels 2, 7 and 9 for the site of L'Aquila in correspondence of the piers, obtained through SRA and relative to  $T_i = 0.5$  s.

### 3.7.3 Performance Levels

In the context of MSA, structural response has been assessed considering three Performance Levels (PL), named Usability Preventing Damage (UPD), Severe Damage (SD) and Structural Failure (SF). Each PL includes different components: Neoprene Pads, Ductile and Brittle Pier Failure Modes, Abutment Backwall and Foundations. UPD level is associated with a light damage condition in the structure, SD level represents an extensive damage condition, and SF includes failure conditions. Table 3.9 lists Engineering Demand Parameters (EDPs) and associated thresholds for each PL.

For neoprene pads,  $d_{fr}$  is the displacement corresponding to the onset of relative sliding between pad and deck,  $d_{60}$  is associated with reaching the limit pressure of 60 MPa on the device and  $d_{ms}$  is the displacement limit for beam unseating. The three displacement thresholds used for bearings are therefore associated with different stages of the response

of unbolted neoprene pad bearings, as described in [31]. For piers, damage is connected to the plastic hinge rotation in case of ductile failures and to crack opening in case of premature shear failures. More specifically,  $\theta_y$  is the chord rotation threshold associated with the yielding point of the pier, while thresholds associated with SD and SF represent conditions where plastic hinge rotation at the base of the piers leads to significant residual displacements. For the abutment backwall, UPD level is associated with closure of the gap between deck and backwall, SD level describes the attainment of the lateral strength of the system and SF level is representative of extensive soil uplift [36]. For foundations, the PLs are described in terms of  $d_y$ ,  $d_d$  and  $d_u$  which represent yielding, significant residual displacements and ultimate rotation of foundation piles. The displacement at the top of foundation piles, described by the 1DMEs response, is used as EDP to characterize the different levels of foundations response. For the single foundation pile, chord rotation in the plastic hinge  $\theta$  corresponding to the three PLs is computed based on the Eurocode formulation, and the corresponding top foundation pile displacement is found consistently with Broms' theory [77][78]. For the sake of clarity, the foundations failure mechanism is represented in Figure 3.13.

### 3.7.4 NLTHA results

The exceedance of the aforementioned PLs is defined computing for each Non-Linear Time History Analysis (NLTHA) the demand to capacity ratio (D/C) associated with each component. For each analysis, the maximum D/C value among the values associated with each component is taken into account to identify the structural performance. Note that this corresponds to assuming that the bridge is a series system, which is a reasonable assumption considering that the case study is a girder bridge with statically determined scheme. For consistency with the approach used in [20][21], viscous damping has been included in NLTHAs through a Rayleigh model with 5% damping ratio at fundamental periods. Despite the possible influence of corrosion on modal damping ratios, demonstrated in studies such as [63], this effect has been neglected. Future developments of this study will investigate the influence of this aspect on the seismic response.

Figures 3.14-3.19 display D/C ratios, number of failures ( $D/C > 1$ ), critical elements and cumulative failures for each PL, for each stripe, considering the structure in pristine and corroded conditions for each of the considered sites. As expected, the number of failures tends to decrease going from UPD to SF performance levels. The influence of seismic hazard on the results is also clear, with an increasing number of failures going from the low seismicity site (Milan) to the high seismicity site (L'Aquila). It is worth noting that, due to the different ground motions used at each IM level in the MSA approach to capture the representative properties of the seismic input, it is possible that strictly increasing number of failures may not be observed [79]. An example of this condition can be observed in Figure 3.15 (c) and Figure 3.15 (i).

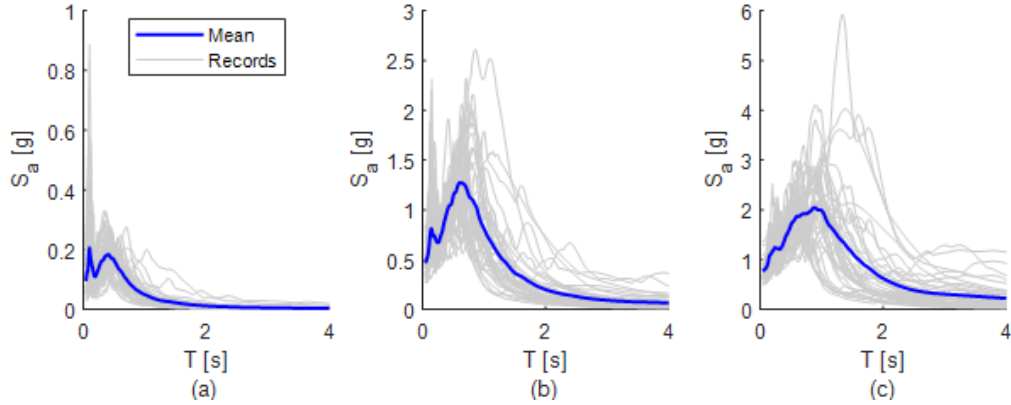


Figure 3.12. 5% damped response spectra of the input motions for IM levels 2 (a), 7 (b) and 9 (c) for the site of L'Aquila in correspondence of the piers, obtained through the SRA.  $T_c = 0.5$  s.

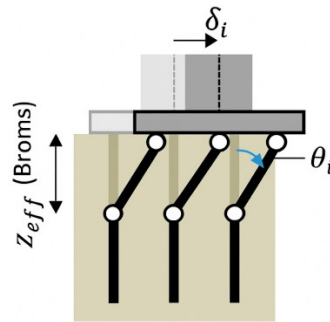


Figure 3.13. Foundations failure mechanism for the  $i$ -th PL. Adapted and modified from [21].

Additional considerations can be made by observing the failure mechanisms characterizing each PL for the site of L'Aquila. In pristine condition, UPD level is dominated by neoprene pad bearings failures, while SD level is strongly influenced by the abutment backwall mechanism for IM levels 1-7 and by the foundation mechanism for IM levels 8-10. Finally, for SF level the predominant failures are in the neoprene pad bearings. A non-negligible effect of foundational failures has been observed for IM levels 8-10. In presence of corrosion, the D/C ratios are slightly higher, but the most important differences have been observed in terms of dominant failures for each PL. UPD is still dominated by neoprene pad bearings failures. For the SD level, abutment backwall mechanism is still dominant for lower IM levels but for high IM levels ductile failures in the bent columns became more relevant. For the SF level, ductile failures in the columns are the most important, with a few foundational failures observed.

The results reveal that corrosion has a more important impact on SD and SF performance levels, which can be explained considering the strong influence of neoprene pad bearings performance on the UPD level, while other mechanisms are more important for SD and SF.

### 3.8 TIME-DEPENDENT SEISMIC RISK ASSESSMENT

To quantify the seismic risk associated with the pristine and corroded structures, the failure rate  $\lambda_f$  is computed for each PL.  $\lambda_f$  can be defined as:

$$(3.14) \quad \lambda_f = \int_0^\infty P(F|IM = x) |d\lambda(IM > x)|$$

In Equation (3.14),  $P(F|IM = x)$  is the fragility function for a given PL and  $|d\lambda(IM > x)|$  is the absolute value of the derivative of the site hazard curve.

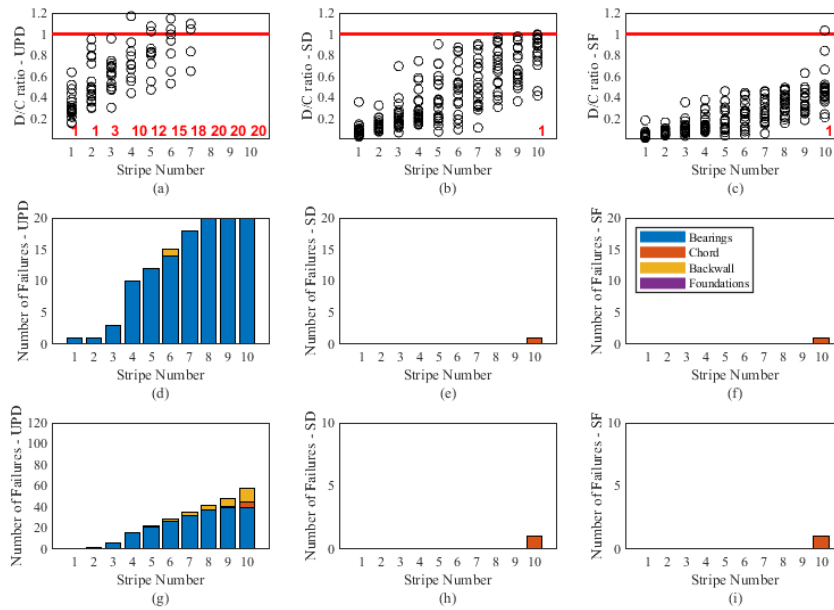


**Figure 3.14.** Structural Response of the bridge in pristine condition, site of Milan. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.



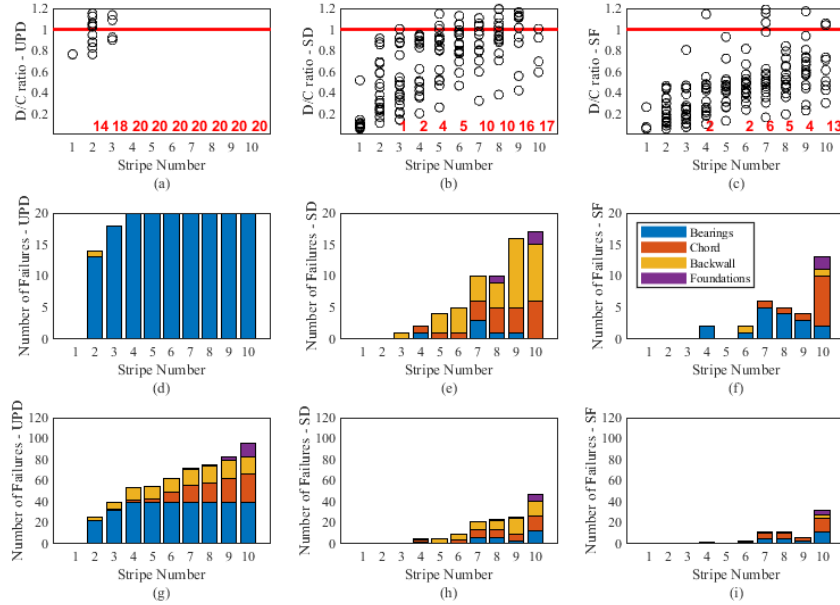


Figure 3.15. Structural Response of the bridge in pristine condition, site of Naples. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.

Table 3.9. EDPs and thresholds for the performance levels, adapted from [21].

| Component         | UPD                                 |            | SD                                  |                                       | SF                                  |             |
|-------------------|-------------------------------------|------------|-------------------------------------|---------------------------------------|-------------------------------------|-------------|
|                   | EDP                                 | Limit      | EDP                                 | Limit                                 | EDP                                 | Limit       |
| Neoprene Pads     | Relative horizontal displacement    | $d_{fr}$   | Relative horizontal displacement    | $d_{60}$                              | Relative horizontal displacement    | $d_{ms}$    |
| Piers (Ductile)   | Chord Rotation                      | $\theta_y$ | Plastic Hinge Rotation              | $\theta_y + 0.5(\theta_u - \theta_y)$ | Plastic Hinge Rotation              | $\theta_u$  |
| Piers (Brittle)   | Shear Force                         | $V_u$      | Shear Force                         | $-0.2V_u$                             | Shear Force                         | $-0.5V_u$   |
| Abutment Backwall | Relative deck-backwall displacement | $Gap$      | Relative deck-backwall displacement | $Gap + F_{bw}/k_{bw}$                 | Relative deck-backwall displacement | $0.1h_{bw}$ |
| Foundations       | Top piles displacement              | $d_y$      | Top piles displacement              | $d_d$                                 | Top piles displacement              | $d_u$       |

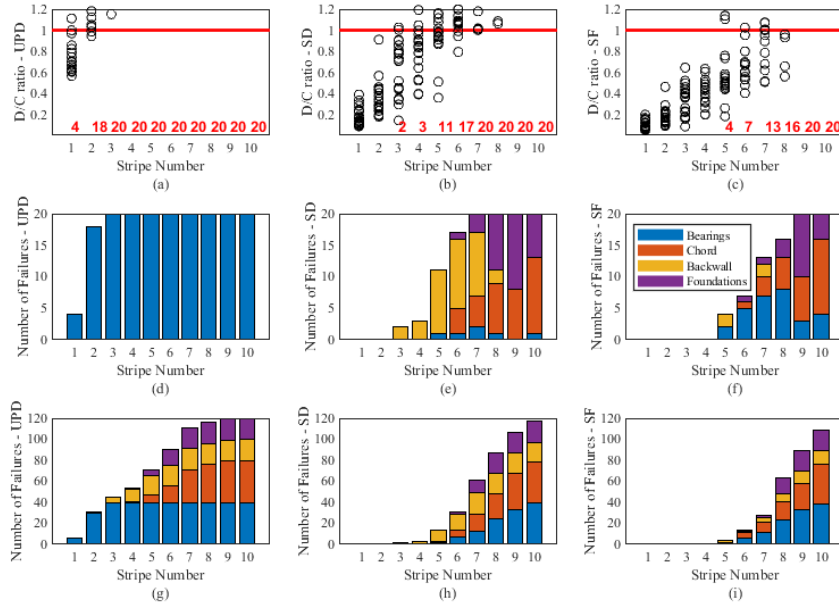


Figure 3.16. Structural Response of the bridge in pristine condition, site of L'Aquila. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.

Fragility functions describe the probability of exceeding a given PL in function of the occurrence of a ground motion with  $IM = x$  and are generally defined as per Equation (3.1):

$$(3.1) \quad P(F|IM = x) = \varphi\left(\frac{\ln\left(\frac{x}{\theta}\right)}{\beta}\right)$$

where  $\varphi()$  is the standard normal cumulative distribution,  $\theta$  is the median value of the fragility function and  $\beta$  is the dispersion of IM. In case of discrete performance levels, like in the present research, the fragility functions related to reaching or exceeding a given PL are defined as:

$$(3.2) \quad P(PL \geq PL_i|IM = x) = \varphi\left(\frac{\ln\left(\frac{x}{\theta_i}\right)}{\beta_i}\right)$$

In Equation (3.2),  $PL_i$  is the specific performance level and  $\theta_i$  and  $\beta_i$  are the fragility parameters estimated for each  $PL$ .

The fragility functions necessary to compute  $\lambda_f$  have been fitted to the results of NLTHAs. The fragility function parameters have been estimated using an EDP-based approach [80], considering the maximum  $D/C$  ratio for each analysis as EDP. For this purpose, the R2R-EU software [81] has been used, employing a Maximum Likelihood Estimation (MLE) method (and a Least-Squares Fitting (LSF) method when MLE was not applicable). Fragility curves for each PL and site, considering pristine and corroded conditions, have been represented in Figure 3.20. Note that curves developed for Milan and Naples have been represented up to an IM level approximately corresponding to MSA IM level 10. It can be observed that including corrosion effects in the assessment leads to a generally more pronounced seismic vulnerability of the bridge, represented by increased probability of exceeding a PL. For the sake of completeness, Table 3.10 reports the fragility function parameters estimated for each site and PL.

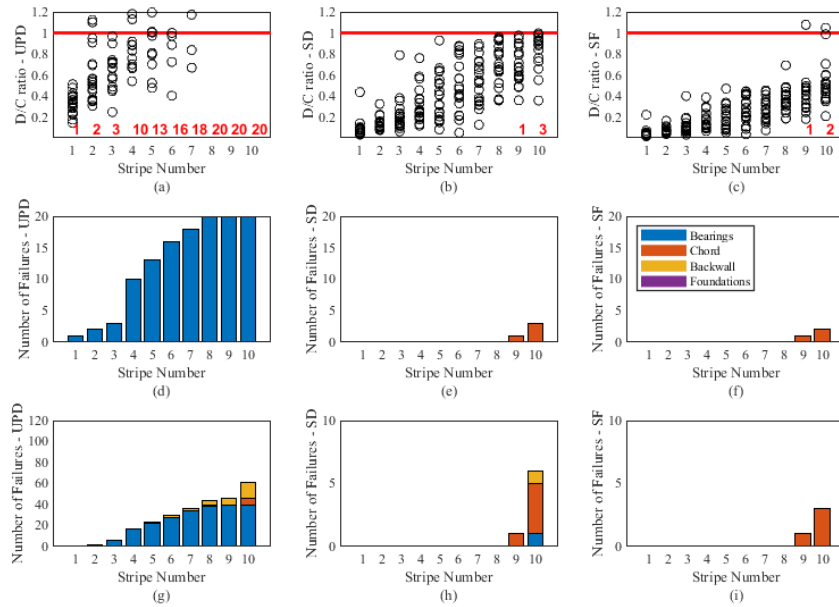


**Figure 3.17. Structural Response of the bridge in corroded condition, site of Milan. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.**

**Table 3.10.** Fragility parameters estimated for each site and PL, in pristine and corroded condition.  
 \*=fragility function fitting not robust enough due to the insufficient number of analyses where the structure did not exceed the performance threshold.

| Site     | Condition | PL  | $\theta$ | $\beta$ |
|----------|-----------|-----|----------|---------|
| Milan    | Pristine  | UPD | 0.063    | 0.542   |
|          |           | SD  | 0.363    | 0.058   |
|          |           | SF  | 1.024    | 0.51    |
|          | Corroded  | UPD | 0.060    | 0.556   |
|          |           | SD  | 0.544    | 0.507   |
|          |           | SF  | 0.686    | 0.608   |
| Naples   | Pristine  | UPD | 0.074    | 0.064   |
|          |           | SD  | 0.575    | 0.768   |
|          |           | SF  | 1.210    | 0.944   |
|          | Corroded  | UPD | 0.072    | 0.474   |
|          |           | SD  | 0.561    | 0.796   |
|          |           | SF  | 0.833    | 0.832   |
| L'Aquila | Pristine  | UPD | 0.069    | 0.380   |
|          |           | SD  | 0.444    | 0.451   |
|          |           | SF  | 0.873    | 0.519   |
|          | Corroded  | UPD | *        | *       |
|          |           | SD  | 0.420    | 0.467   |
|          |           | SF  | 0.773    | 0.501   |

Table 3.11 reports  $\lambda_f$  values for each PL for pristine and corroded conditions and the increase observed in the value considering deterioration phenomena in the analysis. In general terms, the results confirm what has been observed in terms of structural response. Passing from UPD to SF,  $\lambda_f$  decreases while it increases as a function of site seismicity, with higher values observed in L'Aquila and lower in Milan. The results confirm that structural deterioration decreases the structural reliability of the case study in a non-negligible way, with a general increase of  $\lambda_f$  values. The effect is more evident for SD and SF performance levels, while it is less important for the UPD level. This can be explained considering the high vulnerability of the bridge with respect to the former performance level, dominated by neoprene pad bearings failures. It is also worth pointing out that the threshold associated with neoprene pads at UPD level corresponds to the onset of sliding between pad and deck, which may not induce bridge owners or analysts to take immediate action.

Further discussion of the effects of structural deterioration on the considered case study is related to the dominant failure modes at each IML. Along with the increase in  $\lambda_f$  values for each PL, failure mechanisms in the bent columns are more relevant than in pristine condition and the distribution of failures associated with the maximum D/C ratio in each stripe is significantly different, possibly affecting the bridge management decision-making process.

The results reveal that structural deterioration has a non-negligible effect on the performance of the case study.  $\lambda_f$  values associated with the corroded bridge are higher in each site and a modification of structural response and failure modes has been observed, with a possible relevant impact of the managing process of the bridge. Also, not considering the evolution in time of material properties in the assessment process of existing bridge structures is unconservative and may induce underestimation of the associated seismic risk.

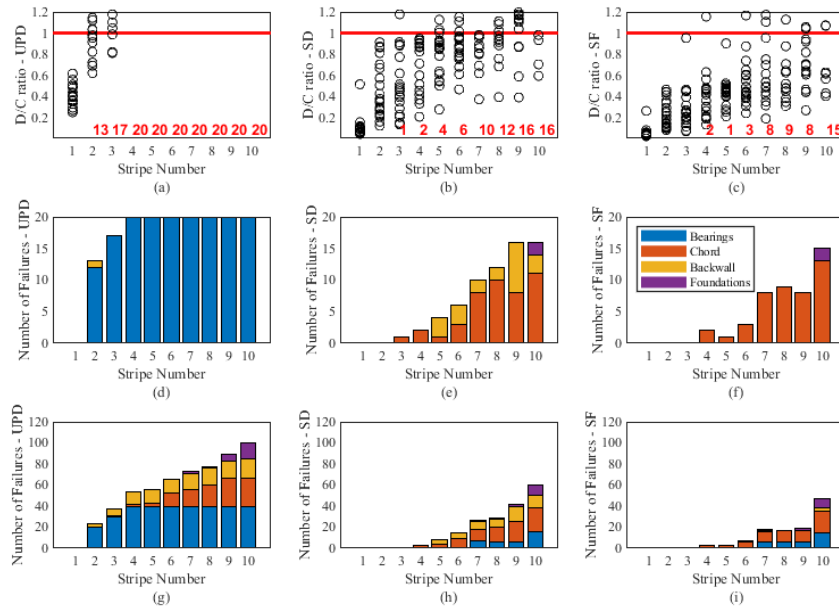


**Figure 3.18.** Structural Response of the bridge in corroded condition, site of Naples. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.

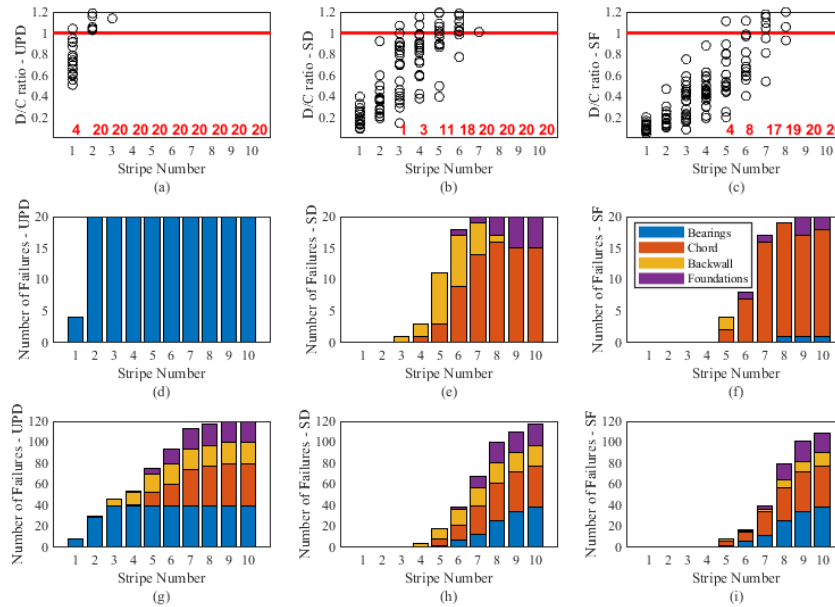


Figure 3.19. Structural Response of the bridge in corroded condition, site of L'Aquila. (a-c) D/C ratios and number of failures for each stripe and Performance Level. (d-f) Failures corresponding to maximum D/C ratio for each stripe and Performance Level. (g-i) Cumulative number of failures for each stripe and Performance Level.

Table 3.11. Failure rate values for each PL and each site in pristine and corroded condition. \*=fragility function fitting not robust enough to compute the corresponding failure rate.

| Site     | PL  | $\lambda_f$ (pristine) | $\lambda_f$ (corroded) | Increase [%] |
|----------|-----|------------------------|------------------------|--------------|
| Milan    | UPD | 1.14E-02               | 1.33E-02               | 16.67        |
|          | SD  | 1.08E-05               | 1.59E-05               | 47.22        |
|          | SF  | 1.18E-06               | 1.32E-05               | 1018.64      |
| Naples   | UPD | 2.43E-02               | 3.08E-02               | 26.75        |
|          | SD  | 1.27E-03               | 1.46E-03               | 14.96        |
|          | SF  | 4.79E-04               | 7.05E-04               | 47.18        |
| L'Aquila | UPD | 6.67E-02               | *                      | -            |
|          | SD  | 2.97E-03               | 3.35E-03               | 12.79        |
|          | SF  | 9.73E-04               | 1.18E-03               | 21.27        |

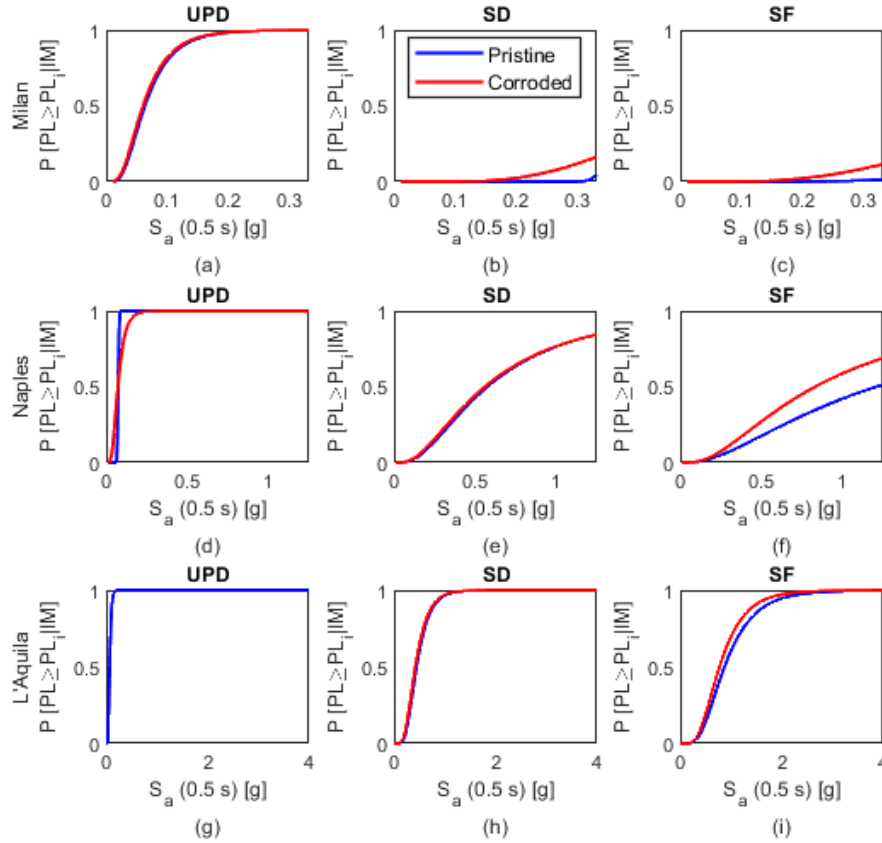


Figure 3.20. Fragility curves for pristine and corroded conditions, for each PL, for the sites of (a-c) Milan, (d-g) Naples and (g-i) L'Aquila.

### 3.9 CONCLUSIONS

This work presented the seismic assessment of a case study representative of existing Italian bridges built in the second part of XX century. The case study has been located in three ideal Italian sites, namely Milan, Naples and L'Aquila, to investigate low, medium and high seismicity conditions. The codes followed in the design have been identified and a simulated design process has been carried out to verify the original design and to adapt the case study to the L'Aquila site. A detailed numerical model including state-of-the-art modeling of foundations and abutments and refined description of neoprene pad bearings response has been developed using the OpenSees platform. A realistic deterioration scenario on bent columns has been identified, implemented and successfully validated through comparison with an extensive experimental campaign conducted on the structure. To study the seismic reliability of the case study in the three sites, MSA has been conducted

considering 10 IM levels and carefully considering the specific site soil conditions through SRA. Three performance levels, UPD, SD and SF have been considered to describe the structural response. Failure rates associated with each PL have been computed and have been considered as a descriptive metric of seismic reliability. The results revealed that the dominant mechanisms for each PL are different. The case study appeared to be particularly vulnerable at the UPD level due to the influence of neoprene pad bearings, confirming the critical role of these components. As expected, considering structural deterioration led to a decrease in the seismic reliability of the case study, particularly for SD and SF levels. For UPD level the effect of structural deterioration was less relevant due to the strong influence of neoprene pads and the inherently high vulnerability already observed in pristine conditions. Remarkably, along with an increase of the  $\lambda_f$  values associated with each PL, modification of structural response and failure modes has been observed, potentially impacting the management process of the bridge in a significant manner. The study also reveals that not considering the evolution of material properties over time in the assessment of existing bridges is unconservative and may lead to underestimating seismic risk.

It is worth noting that the assumptions made in the context of this study may have significant impact on the results. Corrosion damage has been modeled by reducing reinforcement section, steel and concrete mechanical properties. However, the effects of corrosion on facets such as bond-slip and reinforcement buckling have not been included. Both the effects should be included in future developments of the study but have been neglected in the present work due to the lack of specific data for calibration of the numerical model (and the same consideration may be extended to plastic hinge length). Additionally, previous on-field investigations such as [82] highlighted that unbolted neoprene pad bearings tend to create a flexible layer avoiding pier damage during seismic events, justifying the simplifications of the present study. However, the effects of these phenomena should be carefully assessed and, in general, assumptions related to the level of detail of corrosion damage modeling are expected to affect the results of the time-dependent assessment. Future developments of the present work may be related to including the effects of corrosion on bond-slip phenomena and reinforcement buckling, highlighting the relative importance of each damage mechanism in time-dependent assessment of existing bridges. The results also revealed that neoprene pad bearings play a non-negligible role in the seismic response of the case study bridge. As reported in [34], the response of the devices is expected to include a linear viscoelastic branch, followed by frictional behavior and no general consensus on the value of the concrete-neoprene friction coefficient can be found in the literature due to the influence of factors such as applied vertical load and contact surface roughness. In [34], the authors investigated the influence of both  $G$  and  $\mu$  on seismic losses, finding that the variability of the former, due to factors such as aging and creep, does not alter significantly the response of the bridge. In contrast,  $\mu$  exerts a substantially greater influence on seismic-risk metrics, with estimated seismic losses increasing by approximately 75% when  $\mu$  is set at its upper bound rather than its



lower bound. Given this consideration, quantifying the influence of aging on  $\mu$  represents an open research question, indicating the need for future research, both experimental and numerical. Input motions used for the MSA have been selected through a CS-based approach aiming at maintaining a higher order of hazard consistency indirectly accounting for metrics depending on magnitude  $M$  and closest to rupture distance  $R$ , such as duration, but not captured in the response spectra. Given that seismic vulnerability characterization depends on input motions, this aspect should be carefully considered. Extensive investigation on the performance of the approach has been carried out in [69], proving its improved robustness with respect to the original CS approach. Using different input motion selection approaches may lead to slightly different results, but the approach proposed in [69] allows to include in the selection only spectral shapes relevant to the site hazard. Finally, it has to be pointed out that the present investigation refers to a specific case study bridge. Additional investigation, including different bridge archetypes and exposure conditions, is therefore necessary to extend the insights gained in this research.

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## 4. ADVANCED FIBER-BASED MODELING STRATEGY FOR THE SEISMIC ANALYSIS OF CORRODED RC PIERS

*This chapter is extensively based on the manuscript Reale, S., Pavese, A., Palermo, A. “Advanced Fiber-Based Modeling Strategy for the Seismic Analysis of Corroded RC Piers”, (in preparation).*

### 4.1 INTRODUCTION

Corrosion represents the main deterioration phenomenon in Reinforced Concrete (RC) elements, strongly affecting durability and safety of structures over time. The primary effects of corrosion include deterioration of reinforcing bars embedded in concrete, cracking and eventual spalling of concrete due to the expansive pressure of corrosion products, and reduction of the bond strength between steel and concrete. Remarkably, as most existing structures are approaching or have already exceeded their useful lifetime, assessing the capacity of corroded RC elements represents an impelling task [1]. The catastrophic consequences of deterioration and corrosion have been observed in bridge collapses of the past few years, such as the one of the Polcevera Viaduct. The bridge collapsed on August 14, 2018, resulting in 43 fatalities. The bridge exhibited significant deterioration with widespread corrosion [2]. Corrosion also has a significative economic impact, as demonstrated by a study for the Federal Highway Administration revealing that corrosion cost, considering both direct and indirect components, was more than \$550 billion [3].

A number of studies investigated the effects of corrosion on RC elements. Apostolopoulos and Papadopoulos [4] tested the degradation of the tensile properties of steel bars using an accelerated corrosion technique and found that on one hand ductility deterioration is more severe than strength deterioration and on the other hand ductility reduction impacts also the low-cycle fatigue behavior of steel bars. The impact of corrosion on fatigue life is associated with a reduction of the latter, as demonstrated experimentally in [5]. Corrosion directly affects also the inelastic buckling of steel bars, reducing the compressive yield strength [6]. Cracks originated by corrosion products expansion reduce the compressive strength of cover concrete depending on the magnitude of the transverse tensile strain generating microcracks [7][8], as confirmed experimentally in [9]. Only a few studies discuss the influence of corrosion on the properties of confined concrete, addressing the reduced confinement level due to corrosion of transverse reinforcement [10]. Al-Sulaimani et al. [11] investigated the influence of corrosion and cracking on bond behavior, finding that up

to a certain level of corrosion bond strength increases due to the formation of corrosion products, but as corrosion progresses it decreases due to bar deterioration and crack opening. Corrosion influences also failure mode and shear response of RC elements [12], causing in some cases the shift of the failure mode from ductile to brittle [13][14]. The effects of corrosion on the response of RC columns are represented in Figure 4.1.

This chapter aims to propose an efficient and refined fiber-based modeling strategy specifically designed for the seismic analysis of corroded RC piers. The components of the modeling strategy are detailed and discussed in Section 4.2. Section 4.3 presents the validation of the modeling strategy through comparison with quasi-static cyclic tests on both uncorroded and corroded specimens. The performance of the modeling approach is further scrutinized in Section 4.4 through comparison with an alternative less refined approach. Finally, Section 4.5 addresses the influence of corrosion on the dynamic response of RC piers through NTLHAs performed on the benchmark specimens introduced in Section 4.3.

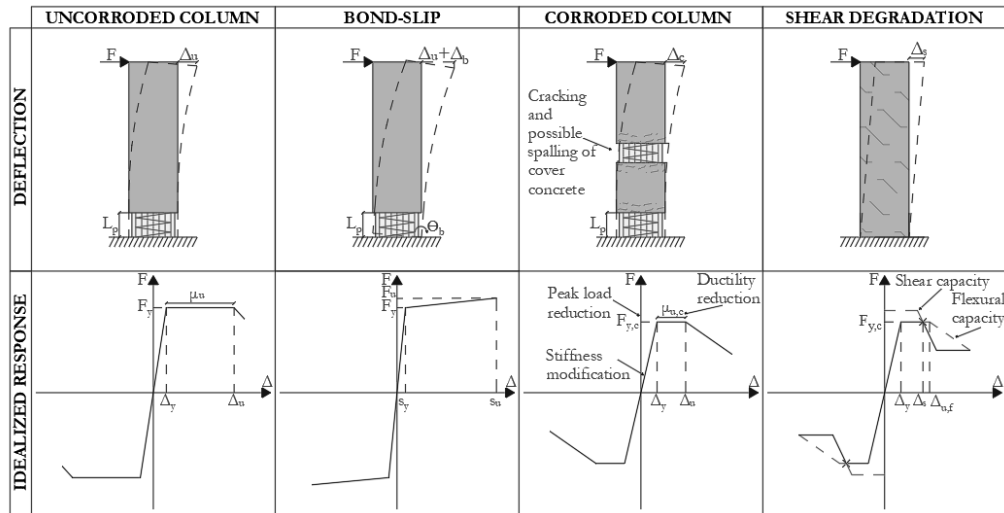


Figure 4.1. Effects of corrosion on the response of RC columns.

## 4.2 MODELING STRATEGY

This chapter presents a numerical model strategy for the seismic simulation of corroded RC piers developed using the OpenSees library [15]. The model captures the flexural response of the elements and includes bond-slip effects at the base of the column but can be readily extended to include the shear response, as in [16], if necessary.

Figure 4.2 represents the concept of the modeling strategy. A force-based nonlinear beam with Gauss-Radau integration scheme is used to avoid localization issues [17]. Inelastic buckling, low-cycle fatigue, bar fractures and concrete crushing are accounted for at material level. A zero-length element with fiber section is used at the base of the element to account for bond-slip effects. It is worth remarking that while the present study considers single column specimens, the modeling strategy can be easily applied to multi-column bents and scaled for modeling complete bridge structures.

#### 4.2.1 Corrosion damage on steel and concrete mechanical properties

The flexural response is captured using a force-based nonlinear beam element with Gauss-Radau integration scheme. The sections of the element are discretized using fibers. The response of concrete fibers can be modeled using the concrete materials available in OpenSees, such as *Concrete04* and *Concrete01*. To take into account inelastic buckling and low-cycle fatigue effects at material level, the *ReinforcingSteel* material can be used. The model has been implemented by Kunnath et al. [18] and successfully used to model the hysteretic behavior of RC columns [19][20]. Bar fractures can be simulated using a *MinMax* criterion.

Inelastic buckling is included in the compressive response using the Gomes and Appleton model [21] and is governed by the slenderness ratio  $\lambda$ , defined as the ratio between the effective length  $L_{eff}$  and the longitudinal bar diameter  $d_b$ .  $L_{eff}$  can be computed as  $n \times s$ , where  $n$  is the buckling mode of the longitudinal reinforcement and  $s$  is the spacing of the transverse reinforcement.  $n$  can be computed applying the iterative method proposed by Dhakal and Maekawa and detailed in [22], based on the simultaneous solution of Equations (4.1)-(4.2):

$$(4.1) \quad \frac{2\pi^4 EI}{n^3 s^3} + \sum_{i=1}^n \frac{c_i K_n}{4} \left(1 - \cos \frac{2i\pi}{n}\right)^2 - \frac{P_n \pi^2}{2ns} = 0$$

$$(4.2) \quad \frac{2\pi^4 EI}{(n+1)^3 s^3} + \sum_{i=1}^{n+1} \frac{c_i K_n}{4} \left(1 - \cos \frac{2i\pi}{n+1}\right)^2 - \frac{P_n \pi^2}{2(n+1)s} = 0$$

where  $EI$  is the average flexural rigidity of the rebar,  $K_n$  is the required stiffness,  $P_n$  is the corresponding load and  $c_i$  is a coefficient that incorporates the plasticity of the lateral ties. Details of formulation and procedure are reported in [22].

Low-cycle fatigue is included in the material formulation using the Coffin-Manson model [23]. Low-cycle fatigue failure is described by Equation (4.3):

$$(4.3) \quad \varepsilon_p = \varepsilon_f (2N_f)^{-\alpha}$$

where  $\varepsilon_p$  is the plastic strain amplitude,  $2N_f$  is the number of half cycles to failure and  $\varepsilon_f$  and  $\alpha$  are material constants.

The presence of corrosion affects the mechanical properties of steel and concrete. The level of corrosion can be quantified through the section loss  $\psi$ , defined as:

$$(4.4) \quad \psi = \frac{A_0 - A_c}{A_0} \cdot 100$$

where  $A_0$  is the uncorroded rebar area and  $A_c$  is the rebar area in corroded condition.

The corrosion-induced deterioration of steel mechanical properties can be described through the models proposed by Du et al. [24][25] (Equations (4.5)-(4.7)):

$$(4.5) \quad f_{y,c} = (1 - 0.005\psi) \cdot f_{y,0}$$

$$(4.6) \quad f_{u,c} = (1 - 0.005\psi) \cdot f_{u,0}$$

$$(4.7) \quad \varepsilon_{su,c} = (1 - 0.05\psi) \cdot \varepsilon_{su,0}$$

where  $f_{y,c}$  is the yield strength in corroded condition,  $f_{y,0}$  is the yield strength in uncorroded condition,  $f_{u,c}$  is the ultimate strength in corroded condition,  $f_{u,0}$  is the ultimate strength in uncorroded condition,  $\varepsilon_{su,c}$  is the ultimate strain in corroded condition and  $\varepsilon_{su,0}$  is the ultimate strain in uncorroded condition. The influence of corrosion on inelastic buckling can be captured by estimating  $L_{eff}$  taking into account corroded bar sections for longitudinal and transverse reinforcement and corroded steel mechanical properties. Despite experimental studies noted a reduction of compressive yield strength of reinforcing bars due to corrosion [26], this phenomenon has not been included due to the limited literature available. Corrosion influences also the fatigue life of steel, as reported in [26]. The  $\alpha$  factor of Coffin-Manson fatigue model is modified according to Equation (4.8):

$$(4.8) \quad \alpha_c = \alpha[1 + 0.004\psi]$$

The compressive strength  $f'_c$  of cover concrete reduces due to the microcracks caused by the expansion of corrosion products. The model of Coronelli and Gambarova [27] is used to model this phenomenon, following the indications of [28] and [29]. The effects of corrosion on confined concrete are strictly connected to the deterioration of transverse reinforcement, leading to a reduction of peak strength  $f'_c$  and ultimate strain  $\varepsilon_{cu}$  [30]. The model proposed in [30], based on [31], is summarized by Equations (4.9)-(4.10):

$$(4.9) \quad f'_c = f_{c0} \left( 2.254 \sqrt{1 + \frac{7.94 f_l^{t,c}}{f_{c0}}} - \frac{2 f_l^{t,c}}{f_{c0}} - 1.254 \right)$$

$$(4.10) \quad \varepsilon_{cu}^c = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{su}^c}{f_c^{t,c}}$$

Where  $f_{c0}$  is the compressive strength of unconfined concrete,  $f_l^{t,c}$  is the effective lateral confining pressure,  $\rho_s$  is the volumetric transverse reinforcement ratio of confined core,  $f_{yh}$  and  $\varepsilon_{su}^c$  are yield strength and ultimate strain of corroded transverse reinforcement.

#### 4.2.2 Bond-Slip

Bond-Slip effects are described at material level using the *BondSP01* material, based on the indications reported in [32]. Corrosion effects on Bond-Slip are addressed modifying the slip at yield stress  $s_y$  according to Equation (4.11):

$$(4.11) \quad S_y^c(mm) = 2.54 \left( \left( \frac{d_b^c(mm)}{8437} \right) \frac{f_{y,c}(MPa)}{\sqrt{f_c'(MPa)}} (2\alpha + 1) \right)^{\frac{1}{\alpha_s}} + 0.34$$

where  $s_y$  is the yielding stress at slip in corroded condition,  $d_b^c$  is the residual diameter of a bar at time  $t$  and  $\alpha_s$  is a parameter taken as 0.4.

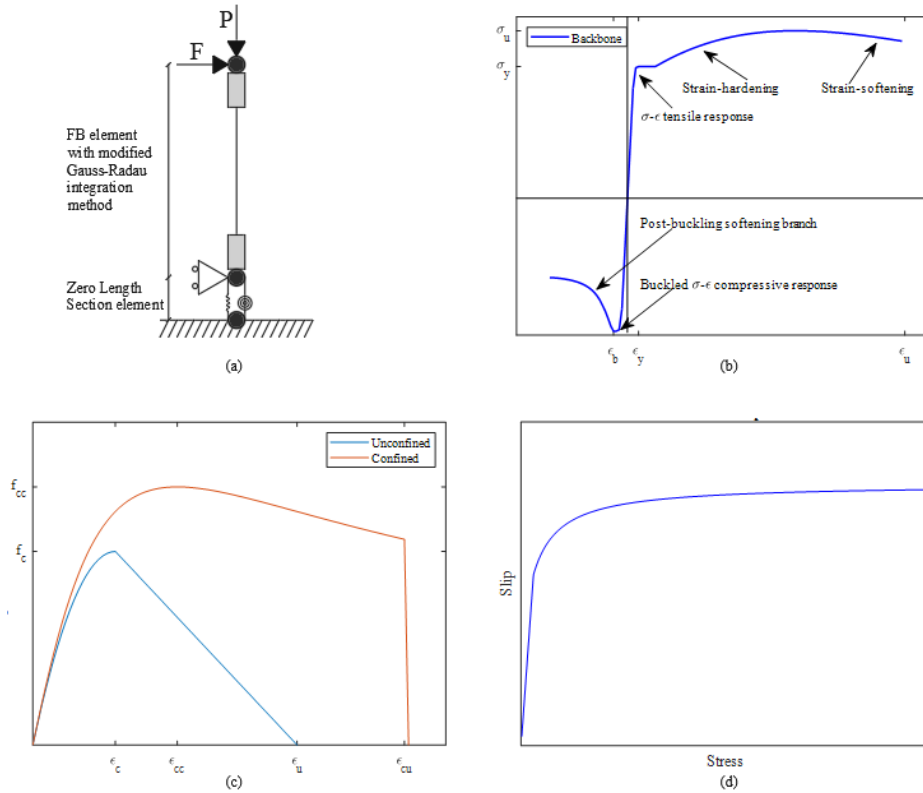


Figure 4.2. (a) Conceptual representation of the modeling strategy, (b) Response of the steel material including buckling, (c) Response of confined and unconfined concrete, (d) Bond-Slip.

### 4.3 MODEL VALIDATION

#### 4.3.1 Benchmark specimens

Model validation is achieved through simulation of three benchmark specimens tested at University of Canterbury [33]. It is worth pointing out that the selected benchmark specimens represent a valuable stress test for the modeling strategy due to the repeated cycles for a range of drift levels, allowing for in-depth evaluation of the performance of the numerical simulations. The three specimens were designed according to NZS3101 [34] with a longitudinal reinforcement ratio of 0.84%. Three different corrosion levels have been investigated: uncorroded (NC), relatively low (CL1) and relatively high (CL2). The characteristics of the specimens are reported in Table 4.1 and the reinforcement layout is represented in Figure 4.3. Grade 300 steel [35] was used for all the specimens. The design compressive strength of the concrete mix was 40 MPa. Concrete samples from each specimen underwent testing on the testing day, revealing a compressive strength of 40 MPa, 53 MPa and 65 MPa for concrete used for NC, CL1 and CL2 respectively.

Accelerated corrosion technique was used to achieve the target corrosion level on the specimens. The measured corrosion levels on longitudinal and transverse reinforcement are included in Table 4.1. The testing protocol was defined following the recommendations reported in [36]. The specimens were subjected to three cycles at increasing drift levels, followed by a half single cycle. Maximum drift levels experienced by the specimens were 6%, 5.5% and 4.5% for NC, CL1 and CL2 respectively. At the end of testing, reinforcement fractures were not detected for NC, while a number of ruptures were observed for CL1 and CL2. The measured cover spalling length was 100 mm (NC), 300 mm (CL1) and 500 mm (CL2), as reported in Figure 4.4.

Table 4.1. Characteristics of the specimens.

| Specimen                          | NC              | CL1             | CL2             |
|-----------------------------------|-----------------|-----------------|-----------------|
| Height [mm]                       | 3100            | 3100            | 3100            |
| Diameter [mm]                     | 500             | 500             | 500             |
| Longitudinal reinforcement        | 8 $\phi$ 16     | 8 $\phi$ 16     | 8 $\phi$ 16     |
| Transverse reinforcement          | $\Phi$ 10/75 mm | $\Phi$ 10/75 mm | $\Phi$ 10/75 mm |
| Longitudinal reinforcement $\psi$ | -               | 5.5             | 10.86           |
| Transverse reinforcement $\psi$   | -               | 12.96           | 27.36           |

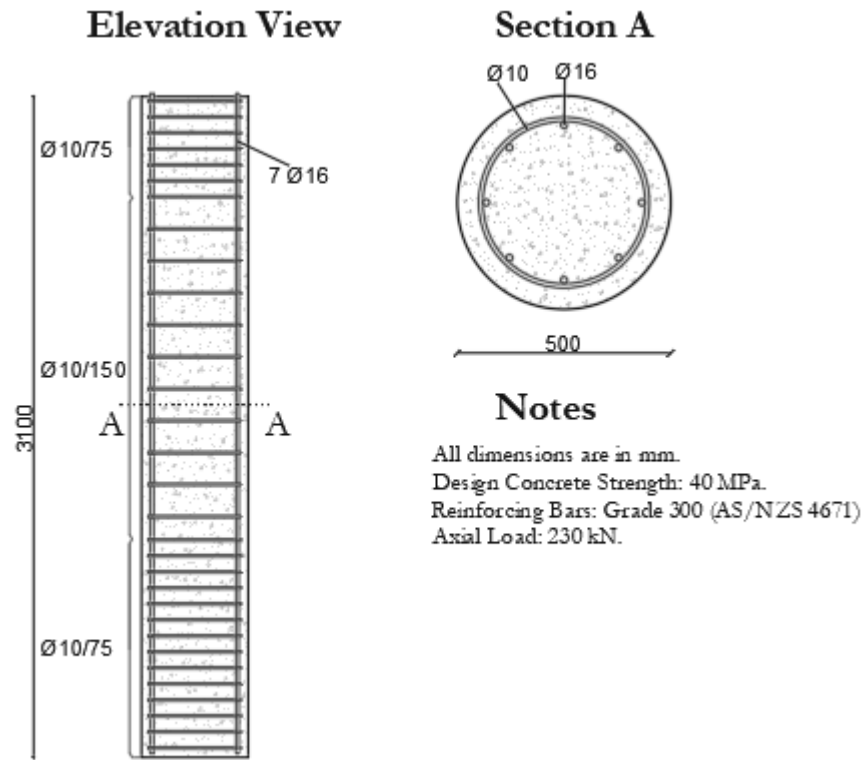


Figure 4.3. Geometry and reinforcement layout of the specimens.

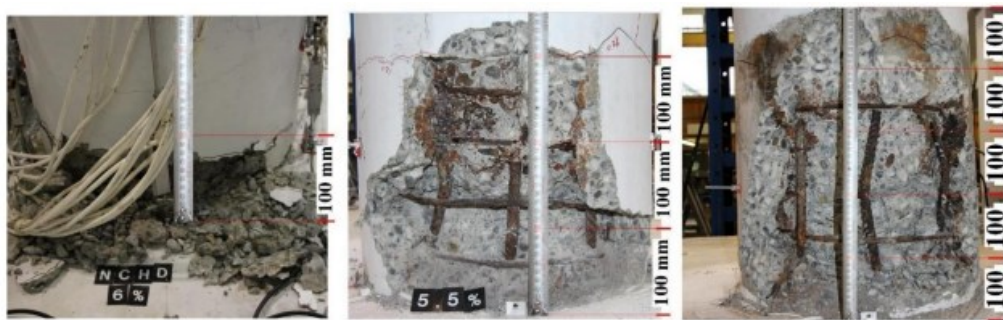


Figure 4.4. Measured cover spalling of the specimens: (a) NC, (b) CL1, (c) CL2. (adapted from [30]).

#### 4.3.2 Simulation with and without inelastic buckling and low-cycle fatigue

A first investigation of the capabilities of the proposed modeling strategy, both in uncorroded and corroded condition, is carried out analyzing the importance of inelastic buckling and low-cycle fatigue for simulating the hysteretic behavior of RC column specimens. The investigation is carried out by comparing the hysteretic response obtained experimentally with the response obtained applying the proposed modeling strategy and the response obtained using a steel uniaxial material that does not include buckling and low-cycle fatigue, such as the *Steel02*, for the simulation of the response.

The results of the comparative simulations, reported in Figure 4.5 and Figure 4.6, reveal that not including inelastic buckling and fatigue leads to overestimating the expected response of the columns, not capturing the cyclic degradation of strength and overestimating the energy dissipated by the element. These considerations can be extended by considering the comparison between some control quantities, namely maximum base shear in positive direction  $V_b^+$  and in negative direction  $V_b^-$ , initial stiffness  $k_{in}$  and equivalent viscous damping  $\xi$ , computed for the two simulation approaches and normalized by their experimental value.  $k_{in}$  has been estimated according to the approach proposed in [37].  $\xi$  has been estimated following the approach proposed by Jacobsen [38].  $\xi$  has been computed for the third cycle at 4% drift for NC and CL1 specimens and for the third cycle at 2% drift for the CL2 specimen. As expected, including inelastic buckling and low-cycle fatigue effects does not affect significantly the estimation of  $V_b^+$  and  $V_b^-$ , which are well captured in both sets of simulations. Consequently, the estimation of  $k_{in}$  yields similar results in both cases. Significant differences are observed in the estimation of the  $\xi$  values. Not including low-cycle fatigue and inelastic buckling leads to systematic overestimation of the  $\xi$  values, which can be explained by the poor representation of the cyclic degradation of the hysteretic response that has been observed experimentally. Remarkably, the  $\xi$  values identified from the numerical simulations applying the proposed modeling strategy were close to the values obtained from the experimental test results of the corroded specimens, demonstrating the capabilities of the modeling strategy to describe realistically the cyclic response of corroded RC members.



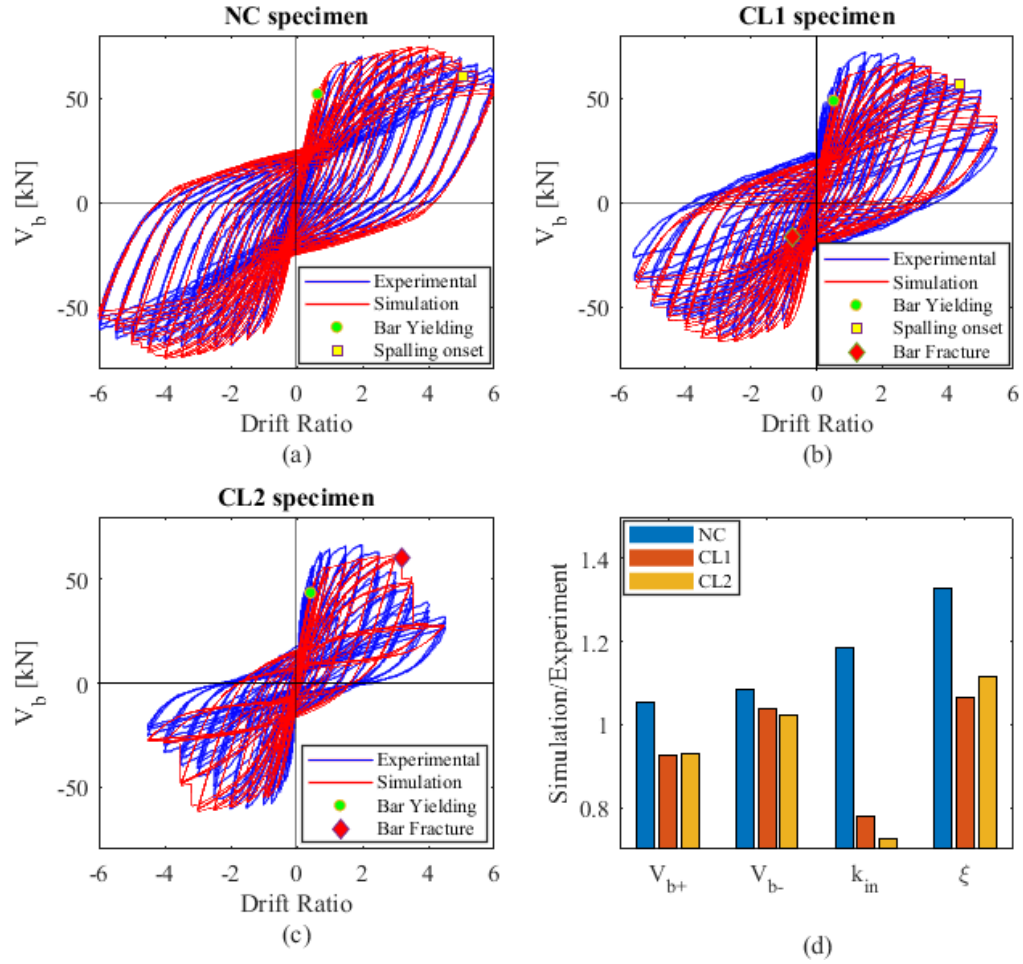


Figure 4.5. Results obtained including inelastic buckling and low cycle fatigue. (a) NC specimen, (b) CL1 specimen, (c) CL2 specimen, (d) normalized control quantities.

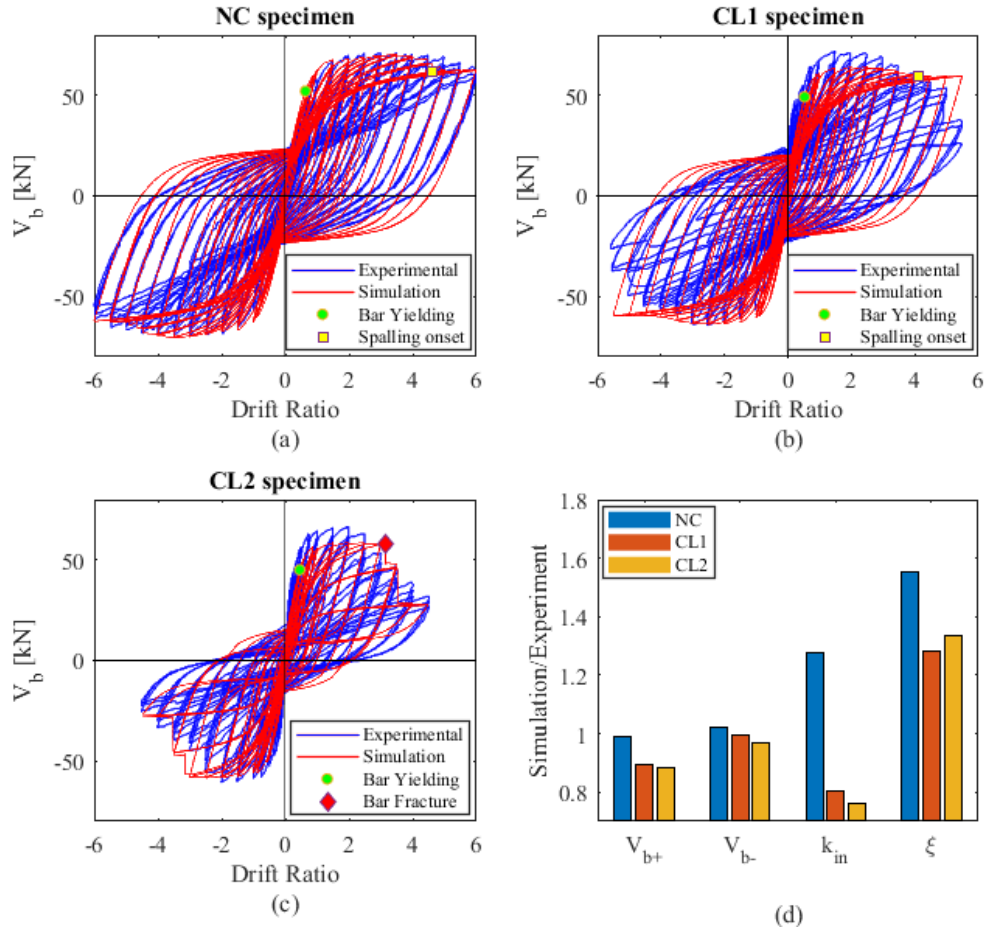


Figure 4.6. Results obtained not including inelastic buckling and low cycle fatigue. (a) NC specimen, (b) CL1 specimen, (c) CL2 specimen, (d) normalized control quantities.

#### 4.3.3 Estimation of the errors of the modeling strategy

The performance of the modeling strategy is further analyzed by comparing the  $\xi$  values obtained from the tests and from the simulations, for each cycle of different drift levels. It is important to remark that estimating correctly the energy dissipation is a critical point of the seismic analysis of RC structures. The results have been reported in Table 4.2, Table 4.3 and Table 4.4. For the NC specimen, the simulation was characterized by a slight overestimation of  $\xi$ . For the CL1 specimen, the numerical results are well aligned with the experimental data. Finally, the results obtained for the CL2 specimen confirm the excellent agreement between models and experimental data. The results confirmed that the numerical modeling strategy proposed in the present work describes adequately the cyclic

response of corroded RC specimens, capturing the cyclic degradation of the hysteretic cycles and providing realistic values of energy dissipation.

**Table 4.2. Comparison between experimental and numerical  $\xi$  values for different drift levels and cycles, NC specimen.**

| NC specimen |                       |                   |                       |                   |                       |                   |
|-------------|-----------------------|-------------------|-----------------------|-------------------|-----------------------|-------------------|
| Drift [%]   | 1 <sup>st</sup> cycle |                   | 2 <sup>nd</sup> cycle |                   | 3 <sup>rd</sup> cycle |                   |
|             | $\xi_{1,exp}$ [%]     | $\xi_{1,num}$ [%] | $\xi_{2,exp}$ [%]     | $\xi_{2,num}$ [%] | $\xi_{3,exp}$ [%]     | $\xi_{3,num}$ [%] |
| 1           | 5.31                  | 4.47              | 2.84                  | 4.70              | 2.61                  | 4.48              |
| 2           | 9.95                  | 13.52             | 8.38                  | 12.43             | 7.6                   | 12.40             |
| 4           | 15.6                  | 20.70             | 15.3                  | 20.23             | 15.2                  | 20.20             |

**Table 4.3. Comparison between experimental and numerical  $\xi$  values for different drift levels and cycles, CL1 specimen.**

| CL1 specimen |                       |                   |                       |                   |                       |                   |
|--------------|-----------------------|-------------------|-----------------------|-------------------|-----------------------|-------------------|
| Drift [%]    | 1 <sup>st</sup> cycle |                   | 2 <sup>nd</sup> cycle |                   | 3 <sup>rd</sup> cycle |                   |
|              | $\xi_{1,exp}$ [%]     | $\xi_{1,num}$ [%] | $\xi_{2,exp}$ [%]     | $\xi_{2,num}$ [%] | $\xi_{3,exp}$ [%]     | $\xi_{3,num}$ [%] |
| 1            | 5.58                  | 5.00              | 2.86                  | 4.91              | 2.63                  | 4.61              |
| 2            | 11.27                 | 14.62             | 9.38                  | 13.58             | 9.03                  | 13.59             |
| 4            | 20.89                 | 22.80             | 20.48                 | 21.85             | 20.36                 | 21.72             |

**Table 4.4. Comparison between experimental and numerical  $\xi$  values for different drift levels and cycles, CL2 specimen.**

| CL2 specimen |                       |                   |                       |                   |                       |                   |
|--------------|-----------------------|-------------------|-----------------------|-------------------|-----------------------|-------------------|
| Drift [%]    | 1 <sup>st</sup> cycle |                   | 2 <sup>nd</sup> cycle |                   | 3 <sup>rd</sup> cycle |                   |
|              | $\xi_{1,exp}$ [%]     | $\xi_{1,num}$ [%] | $\xi_{2,exp}$ [%]     | $\xi_{2,num}$ [%] | $\xi_{3,exp}$ [%]     | $\xi_{3,num}$ [%] |
| 1            | 8.24                  | 6.37              | 7.53                  | 6.12              | 5.24                  | 5.95              |
| 2            | 14.11                 | 15.65             | 13.12                 | 14.43             | 12.92                 | 14.42             |

#### 4.4 COMPARISON WITH AN ALTERNATIVE STRATEGY

The performance of the proposed modeling strategy is compared with that obtained using an alternative strategy for corrosion damage modeling, less refined, retrieved from the literature [39]. This comparison is oriented towards a qualitative evaluation of the differences in the results obtained in the numerical simulations considering a different level of detail. While the results cannot be considered universally valid, they are valuable for

providing insights into the implications of corrosion damage modeling on the results of structural analysis including deterioration phenomena.

#### 4.4.1 Alternative modeling strategy

The Alternative corrosion damage modeling strategy, retrieved from [39], represents a less refined approach with respect to the one proposed in this work and is considered for this comparative investigation due to the similarities with the proposed approach. Deterioration of steel properties only includes reduction of  $f_y$  and  $f_u$  according to Equations (4.5)-(4.6). Cover concrete compressive strength reduction is computed as per [27], while core concrete compressive strength is modified according to Equation (4.9). A summary of the corrosion damage included in each approach is reported in Table 4.5.

Table 4.5. Comparison between Refined and Alternative corrosion damage modeling approaches.

| Corrosion Damage   | Refined Approach | Alternative Approach |
|--------------------|------------------|----------------------|
| $f_y$              | Included         | Included             |
| $f_u$              | Included         | Included             |
| $\varepsilon_{su}$ | Included         | Not included         |
| $f_c$              | Included         | Included             |
| $f'_c$             | Included         | Included             |
| $\varepsilon_{cu}$ | Included         | Not included         |
| $\alpha$           | Included         | Not included         |
| $L_{eff}$          | Included         | Not included         |
| $s_y$              | Included         | Not included         |

#### 4.4.2 Comparison with the refined strategy

The comparison between Refined and Alternative corrosion damage modeling approaches has been carried out by simulating the corroded specimens simulated in Section 4.3. The results of the simulations carried out using the Alternative modeling approach are represented in Figure 4.7. The Alternative approach is not able to capture the cyclic degradation of the response of the specimens. The values of  $V_b^+$  and  $V_b^-$  are well described, but severe overestimation of  $k_{in}$  has been observed for the NC specimen. As expected, not capturing the cyclic degradation of the hysteretic response led to overestimating the  $\xi$  values. The results confirm that the choice of corrosion damage modeling approaches is fundamental in the framework of life-cycle assessment of deteriorating structures, highlighting visible differences in the hysteretic response that could lead to different results in the life cycle metrics. A quantitative description of the results can be found in Table 4.6, where the normalized control quantities obtained for the two modeling approaches have been reported.

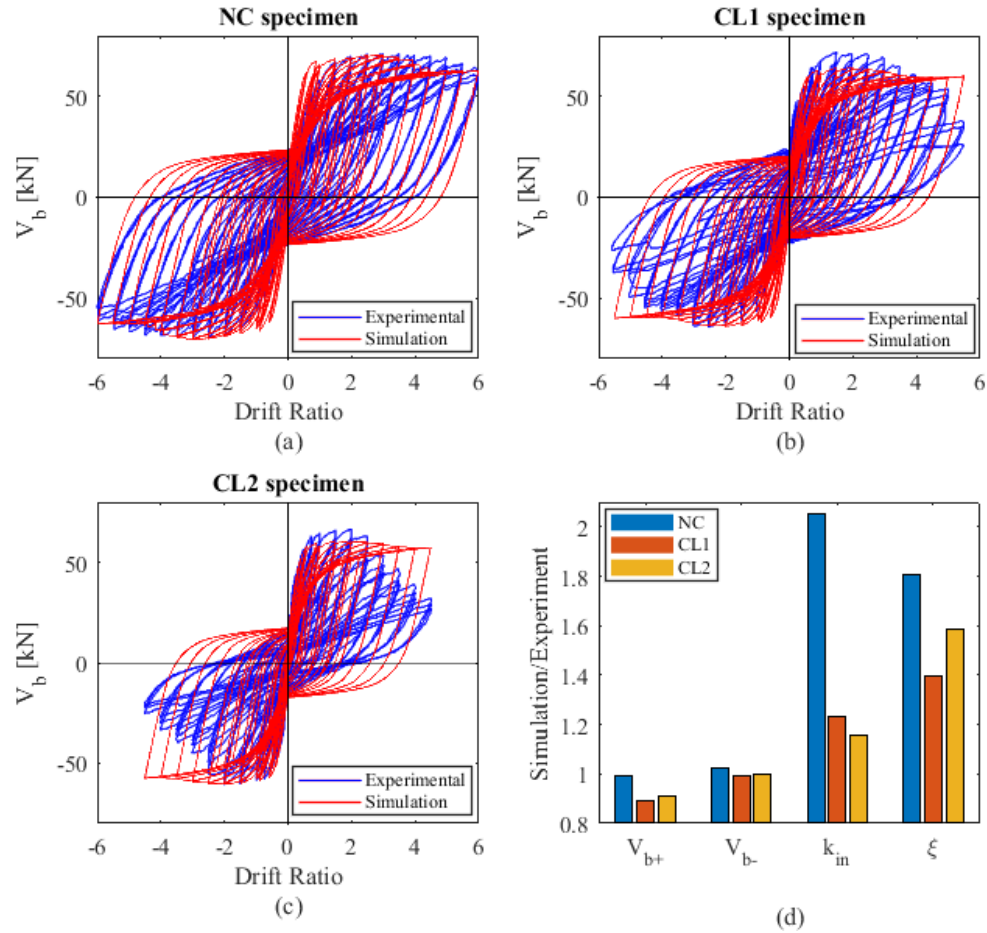


Figure 4.7. Results using the Alternative approach. (a) NC specimen, (b) CL1 specimen, (c) CL2 specimen, (d) normalized control quantities.

#### 4.5 EFFECT OF CORROSION ON THE SEISMIC RESPONSE OF RC PIERS

The seismic response of corroded RC piers is simulated using the proposed modeling approach for NLTHAs. The specimens simulated in Section 4.3 are considered for this investigation. Two different ground motion sets have been considered. To provide further insights into the implications of modeling assumptions, Refined and Alternative corrosion damage modeling approaches have both been considered.

Table 4.6. Normalized control quantities for the three specimens, using the Refined and the Alternative approach.

| Specimen | Refined approach |          |          |       | Alternative approach |          |          |       |
|----------|------------------|----------|----------|-------|----------------------|----------|----------|-------|
|          | $V_{b+}$         | $V_{b-}$ | $k_{in}$ | $\xi$ | $V_{b+}$             | $V_{b-}$ | $k_{in}$ | $\xi$ |
| NC       | 1.05             | 1.09     | 1.19     | 1.33  | 0.99                 | 1.02     | 2.05     | 1.81  |
| CL1      | 0.93             | 1.04     | 0.78     | 1.07  | 0.89                 | 0.99     | 1.23     | 1.40  |
| CL2      | 0.93             | 1.02     | 0.73     | 1.12  | 0.91                 | 1.00     | 1.16     | 1.50  |

#### 4.5.1 Seismic input

Two ground motion sets have been selected for the simulation of the seismic response of corroded RC columns. The first set (Set No. 1) has been selected and scaled to match the 975 years return period response spectrum defined by the Italian Code [40] for the site of L'Aquila (Italy). The selection has been limited to records with magnitude  $M$  between 4.5 and 7.0 and distance  $R$  between 0 and 20 km recorded on rock soil, consistently with the seismic hazard deaggregation. The main characteristics of the ground motions of Set No. 1 are reported in Table 4.7 and the response spectra are represented in Figure 4.8. The second suite (Set No. 2) consists of seven unscaled triplets taken from set No. 3 defined in a PEER project [41]. The set includes records with strong velocity pulses in their strike-normal component. The main characteristics of the records of Set No. 2 are reported in Table 4.8.

Table 4.7. Selected ground motions for GM Set No. 1.

| GM number | Event                        | Station                 | Year | $M_w$ |
|-----------|------------------------------|-------------------------|------|-------|
| 1         | Friuli 2 <sup>nd</sup> shock | S. Rocco                | 1976 | 5.9   |
| 2         | Friuli 3 <sup>rd</sup> shock | S. Rocco                | 1976 | 6.0   |
| 3         | Central Italy                | Monte Fiegni<br>Fiastra | 2016 | 5.9   |
| 4         | Southern Italy               | Viggianello             | 1998 | 5.6   |
| 5         | Sicily Italy                 | Santa Venerina          | 2018 | 4.9   |
| 6         | Central Italy                | Avendita                | 2016 | 5.9   |
| 7         | Caserta                      | Valle Agricola          | 2013 | 5.0   |

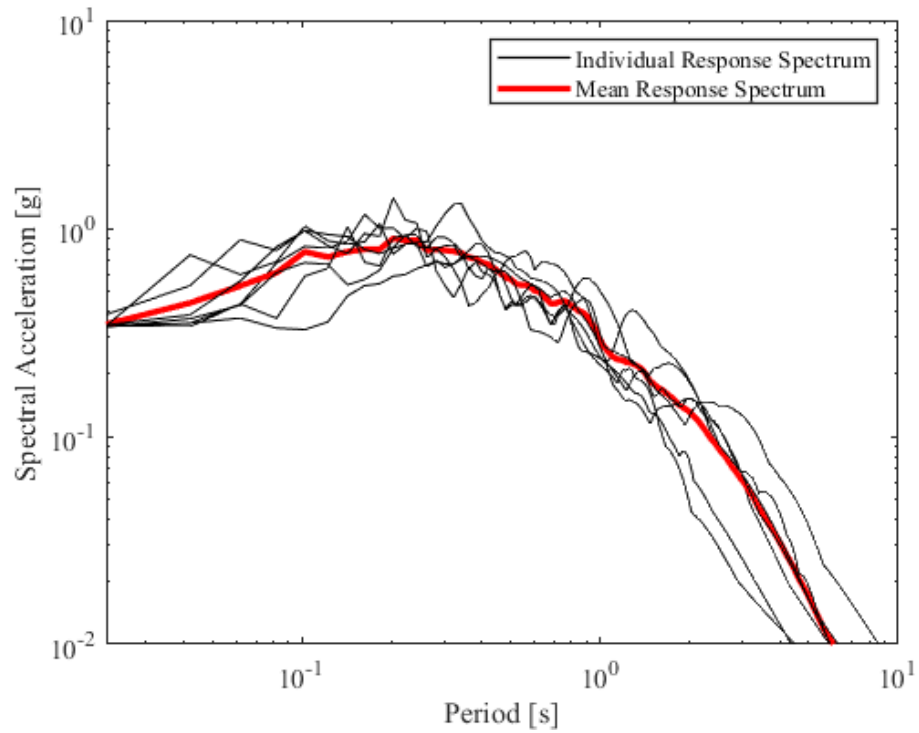


Figure 4.8. Response Spectra for Set No. 1.

Table 4.8. Selected ground motions for GM Set No. 2.

| GM number | Event              | Station                   | Year | $M_w$ |
|-----------|--------------------|---------------------------|------|-------|
| 8         | Imperial Valley-06 | EC Meloland Overpass FF   | 1979 | 6.53  |
| 9         | Morgan Hill        | Coyote Lake Dam (SW Abut) | 1984 | 6.19  |
| 10        | Loma Prieta        | LGPC                      | 1989 | 6.93  |
| 11        | Landers            | Lucerne                   | 1992 | 7.28  |
| 12        | Northridge-01      | Rinaldi Receiving Station | 1994 | 6.69  |
| 13        | Kobe, Japan        | KJMA                      | 1995 | 6.90  |
| 14        | Chi-Chi, Taiwan    | TCU052                    | 1999 | 7.62  |

#### 4.5.2 NLTHAs

For Set No.1, two different metrics are considered for the investigation: the maximum drift level experienced for a given record and the post-event functionality. The latter, considered only for corroded specimens in order to assess the impact of different damage modeling approaches, is evaluated in this way [42]:

- The pre-event available ductility  $\mu$  is computed through pushover analysis;
- The specimen is subjected to NLTHA;
- The post-event available ductility  $\mu_{pe}$  is computed through pushover analysis of the specimen in its post-earthquake configuration;
- The functionality loss  $L$  is computed as per Equation (4.12).

$$(4.12) \quad L = \frac{\mu - \mu_{pe}}{\mu} \cdot 100 [\%]$$

Figure 4.9 shows the seismic response of the three specimens for record No. 6 of Set No.1, considering the two corrosion damage modeling approaches. It can be observed that the Alternative approach systematically leads to lower drift levels in the simulations. The hysteretic response obtained using the two approaches is significantly different, confirming that the assumptions related to corrosion damage modeling affect the results of the simulations. The peak base shear experienced by the specimens modeled using the two approaches is similar, confirming the results obtained in the previous sections.

The maximum drift levels experienced by the specimens are reported in Figure 4.10-Figure 4.12. The results show that, generally, the Alternative corrosion damage modeling approach underestimates the maximum drift level experienced by the column. This can be explained considering that the Refined approach includes bond-slip effects, which are necessary to not underestimate the displacement level of RC members in seismic condition.

The values of  $\mu_{pe}$  and  $L$  for CL1 and CL2 specimens are reported in Table 4.9 and Table 4.10. The Alternative approach is associated with significantly higher  $\mu$  values, both in pre- and post-event conditions, with respect to the Refined approach. The differences in the damage modeling approaches explain this discrepancy. The Refined approach considers the reduction of  $\varepsilon_{su}$  and  $\varepsilon_{cu}$  which directly translates in more limited available ductility in corroded condition. While the results are comparable in terms of  $L$ , the Alternative approach overestimates the available functionality in post-event condition in absolute terms.



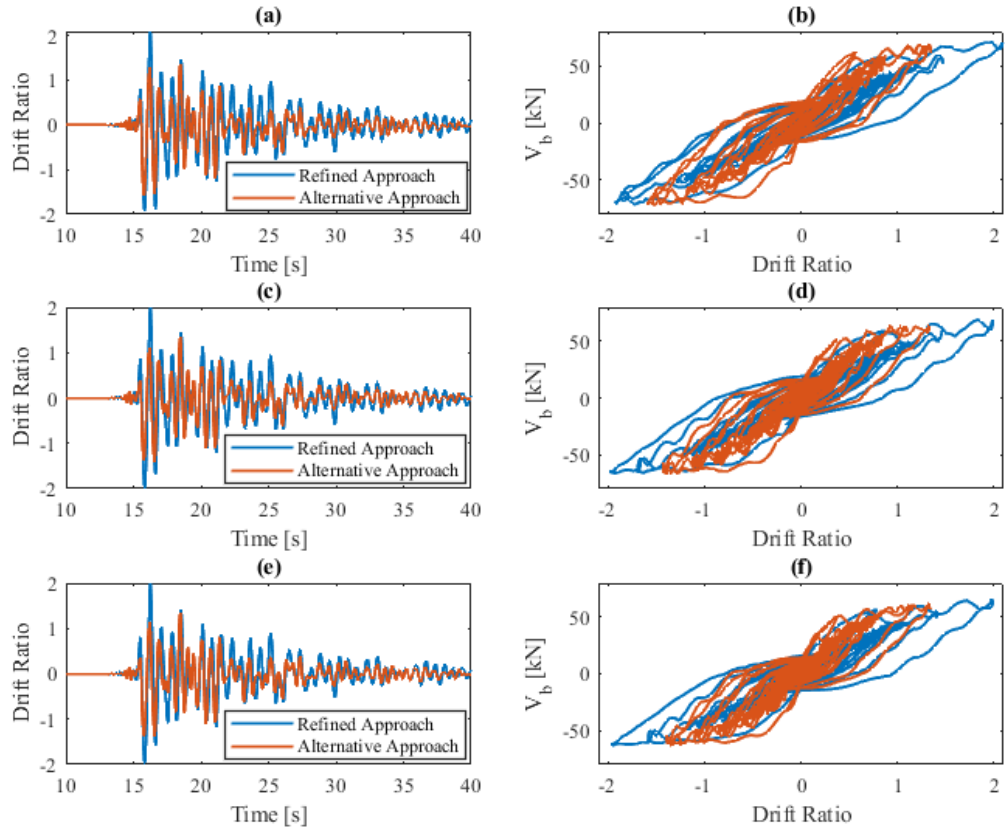


Figure 4.9. Seismic response of the specimens for GM No. 6 of Set No.1. (a-b) NC specimen, (c-d) CL1 specimen, (e-f) CL2 specimen.

Table 4.9. Post event functionality for the two approaches and the CL1 specimen, GM Set No. 1.

| CL1                | Refined Approach |           |       | Alternative Approach |       |           |       |
|--------------------|------------------|-----------|-------|----------------------|-------|-----------|-------|
| Initial $\mu$ [-]  | 9.34             | L [%]     | 0.00  | Initial $\mu$ [-]    | 20.32 | L [%]     | 0.00  |
| $\mu_{pe}$ GM1 [-] | 7.17             | L GM1 [%] | 23.23 | $\mu_{pe}$ GM1 [-]   | 13.90 | L GM1 [%] | 31.57 |
| $\mu_{pe}$ GM2 [-] | 7.01             | L GM2 [%] | 24.92 | $\mu_{pe}$ GM2 [-]   | 11.22 | L GM2 [%] | 44.79 |
| $\mu_{pe}$ GM3 [-] | 6.77             | L GM3 [%] | 27.53 | $\mu_{pe}$ GM3 [-]   | 11.74 | L GM3 [%] | 42.22 |
| $\mu_{pe}$ GM4 [-] | 5.69             | L GM4 [%] | 39.09 | $\mu_{pe}$ GM4 [-]   | 13.06 | L GM4 [%] | 35.69 |
| $\mu_{pe}$ GM5 [-] | 4.60             | L GM5 [%] | 50.76 | $\mu_{pe}$ GM5 [-]   | 9.46  | L GM5 [%] | 53.42 |
| $\mu_{pe}$ GM6 [-] | 4.23             | L GM6 [%] | 54.75 | $\mu_{pe}$ GM6 [-]   | 9.20  | L GM6 [%] | 54.72 |
| $\mu_{pe}$ GM7 [-] | 5.22             | L GM7 [%] | 44.12 | $\mu_{pe}$ GM7 [-]   | 11.91 | L GM7 [%] | 41.39 |

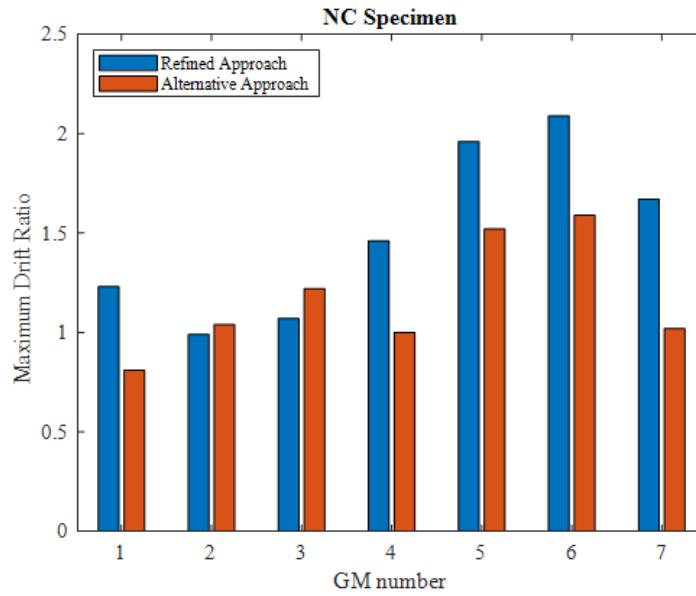


Figure 4.10. Maximum Drift levels experienced by the NC specimen for GM set No. 1.

Table 4.10. Post event loss of functionality for the two approaches and the CL2 specimen, GM Set No. 2.

| CL2                | Refined Approach |           |       | Alternative Approach |       |           |       |
|--------------------|------------------|-----------|-------|----------------------|-------|-----------|-------|
| Initial $\mu$ [-]  | 4.40             | L [%]     | 0.00  | Initial $\mu$ [-]    | 21.91 | L [%]     | 0.00  |
| $\mu_{pe}$ GM1 [-] | 3.23             | L GM1 [%] | 26.58 | $\mu_{pe}$ GM1 [-]   | 14.32 | L GM1 [%] | 34.63 |
| $\mu_{pe}$ GM2 [-] | 3.10             | L GM2 [%] | 29.58 | $\mu_{pe}$ GM2 [-]   | 11.35 | L GM2 [%] | 48.18 |
| $\mu_{pe}$ GM3 [-] | 3.10             | L GM3 [%] | 29.70 | $\mu_{pe}$ GM3 [-]   | 12.80 | L GM3 [%] | 41.60 |
| $\mu_{pe}$ GM4 [-] | 2.55             | L GM4 [%] | 42.03 | $\mu_{pe}$ GM4 [-]   | 13.42 | L GM4 [%] | 38.77 |
| $\mu_{pe}$ GM5 [-] | 2.09             | L GM5 [%] | 52.52 | $\mu_{pe}$ GM5 [-]   | 9.68  | L GM5 [%] | 55.82 |
| $\mu_{pe}$ GM6 [-] | 1.93             | L GM6 [%] | 56.28 | $\mu_{pe}$ GM6 [-]   | 9.95  | L GM6 [%] | 54.59 |
| $\mu_{pe}$ GM7 [-] | 2.33             | L GM7 [%] | 47.16 | $\mu_{pe}$ GM7 [-]   | 12.71 | L GM7 [%] | 41.98 |

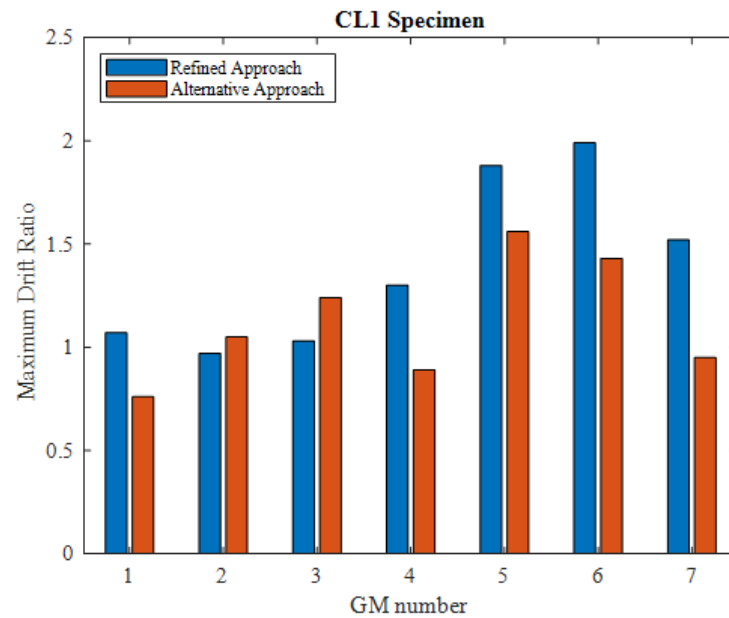


Figure 4.11. Maximum Drift levels experienced by the CL1 specimen for GM set No. 1.

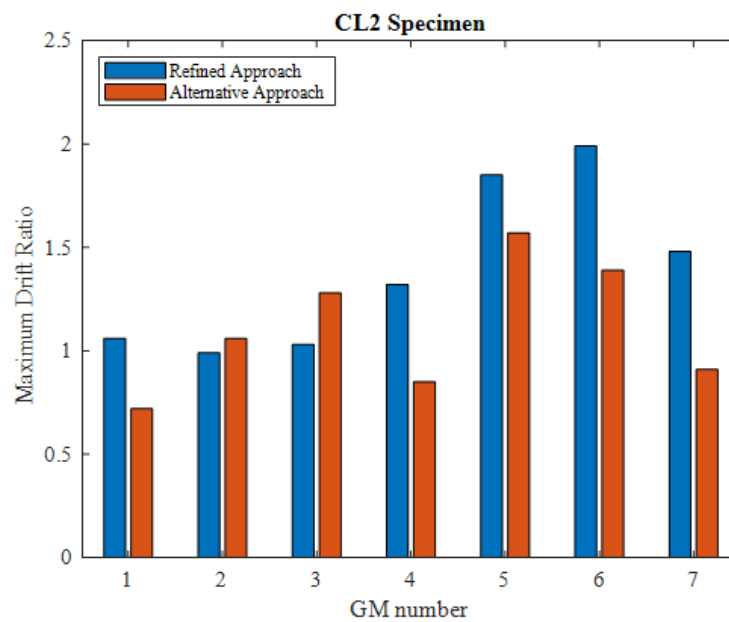


Figure 4.12. Maximum Drift levels experienced by the CL2 specimen for GM set No. 1.

For GM set No. 2, results are presented considering the maximum drift level in longitudinal and transverse direction, denoted as  $d_x$  and  $d_y$  respectively, and the residual drift  $d_{res}$ .  $d_{res}$  is computed combining the residual drifts in the two direction (denoted as  $d_{res,x}$  and  $d_{res,y}$ ), as per Equation (4.13):

$$(4.13) \quad d_{res} = \sqrt{d_{res,x}^2 + d_{res,y}^2}$$

The results for CL1 and CL2 are reported in Table 4.11 and Table 4.12. The results obtained in terms of maximum drift level obtained for Set No. 1 have been confirmed, the Alternative approach tends to underestimate the maximum drift level, in some cases in non-negligible way. No precise trend has been detected in terms of  $d_{res}$ , indicating the need for future research.

**Table 4.11. Results for GM set No. 2 for the two approaches, specimen CL1.**

| CL1 | Refined Approach |           |               | Alternative Approach |           |               |
|-----|------------------|-----------|---------------|----------------------|-----------|---------------|
| GM  | $d_x$ [%]        | $d_y$ [%] | $d_{res}$ [%] | $d_x$ [%]            | $d_y$ [%] | $d_{res}$ [%] |
| 8   | 1.37             | 0.39      | 0.03          | 1.25                 | 0.39      | 0.03          |
| 9   | 0.20             | 0.32      | 0.01          | 0.25                 | 0.35      | 0.02          |
| 10  | 1.78             | 1.04      | 0.07          | 1.43                 | 0.71      | 0.11          |
| 11  | 1.31             | 0.46      | 0.00          | 0.81                 | 0.32      | 0.05          |
| 12  | 0.82             | 1.01      | 0.12          | 0.82                 | 0.74      | 0.07          |
| 13  | 0.63             | 0.30      | 0.01          | 0.63                 | 0.32      | 0.02          |
| 14  | 1.59             | 1.29      | 0.07          | 1.44                 | 1.32      | 0.03          |

**Table 4.12. Results for GM set No. 2 for the two approaches, specimen CL2.**

| CL2 | Refined Approach |           |               | Alternative Approach |           |               |
|-----|------------------|-----------|---------------|----------------------|-----------|---------------|
| GM  | $d_x$ [%]        | $d_y$ [%] | $d_{res}$ [%] | $d_x$ [%]            | $d_y$ [%] | $d_{res}$ [%] |
| 8   | 1.34             | 0.39      | 0.04          | 1.20                 | 0.37      | 0.04          |
| 9   | 0.23             | 0.35      | 0.05          | 0.25                 | 0.35      | 0.02          |
| 10  | 1.69             | 1.01      | 0.05          | 1.43                 | 0.70      | 0.10          |
| 11  | 1.51             | 0.46      | 0.02          | 0.81                 | 0.32      | 0.05          |
| 12  | 0.78             | 0.98      | 0.12          | 0.81                 | 0.73      | 0.07          |
| 13  | 0.65             | 0.29      | 0.00          | 0.60                 | 0.29      | 0.01          |
| 14  | 1.69             | 1.29      | 0.09          | 1.38                 | 1.31      | 0.03          |

#### 4.6 CONCLUSIONS

A refined corrosion damage modeling approach specifically designed for the seismic analysis of RC bridge piers has been presented. The fiber-based modeling strategy aims to represent a practical tool for refined numerical analysis including inelastic buckling of rebars, low-cycle fatigue, and bond-slip effects. The components of the modeling strategy, including state-of-the-art deterioration models have been presented and discussed. The approach has been validated through the simulation of corroded and uncorroded specimens subjected to quasi-static cyclic tests at University of Canterbury. The simulations have been able to capture realistically the hysteretic response of the specimens, describing correctly the cyclic degradation of the hysteretic response. The effects of inelastic buckling and low-cycle fatigue on the results have been investigated through comparison with simulations not including these effects, revealing that including them is fundamental to capture the degradation of the member response. The performance of the modeling strategy has been further investigated by computing the  $\xi$  values associated with different drift levels for each cycle, confirming the good agreement with the experimental results. An Alternative approach has been investigated, and the results of the simulation carried out by applying it have been compared with those of the Refined approach, revealing that both the approaches are able to estimate correctly  $V_b$ , while the latter is more effective for describing the cyclic response of corroded RC members and all the related metrics. NLTHAs have been carried out using both Refined and Alternative approach. The results of the NLTHAs revealed that the Alternative approach underestimates the maximum drift level experienced by specimens and overestimates the available ductility in presence of corrosion, both in the pre- and post- event phase. A precise trend in terms of  $d_{res}$  has not been established, indicating the need for future research in order to understand the impact of damage modeling on this quantity.

The approach is oriented towards practical implementation and use for experimental test design and research in the life cycle engineering field. The approach is modular and could be readily updated over time once that more experimental data are available.

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## 5. MUTUAL INTERACTION BETWEEN ENVIRONMENTAL AND SEISMIC HAZARD AND IMPLICATIONS ON DURABILITY AND LIFE CYCLE SEISMIC ASSESSMENT OF RC BRIDGES

### 5.1 INTRODUCTION

Predictive models currently used for Life Cycle Assessment of RC structures do not consider the influence of cracks on cover concrete on corrosion initiation and propagation. Cracks represent preferential paths for aggressive agents (such as chlorides) to enter the concrete matrix, significantly affecting the progression of corrosion over time. The effects of cracks on corrosion initiation and propagation have been extensively studied over the years, aiming at overcoming the limitations of current corrosion evolution models.

In uncracked concrete, a passive layer on the steel surface prevents corrosion, which starts after its destruction. On the contrary, the presence of cracks allows rapid chloride penetration, and the passive film is never produced [1]. Corrosion initiation is significantly affected by cracks, but no correlation between crack width and critical chloride content  $C_{crit}$  has been found. The influence of cracks on corrosion propagation and on mitigation effects is also an open debate [1]. The finding that cracks may lead to accelerating the corrosion process with respect to uncracked concrete is typical of laboratory studies, that do not include crack healing effects that may occur over time and reduce corrosion rate. Also, from the chemical point of view, the kinetics of corrosion in cracked concrete is controlled by oxygen and moisture transport in the uncracked cover concrete, and thus the corrosion rate may be similar to corrosion in uncracked concrete [2].

Lu et al. [3] proposed a reliability-based approach to predict corrosion initiation time  $t_i$  in RC cracked beams exposed to de-icing salts and developed a model to estimate chloride diffusivity in cracked concrete based on the Eurocode 2 model for cracks [4] and on the formulation proposed by Djerbi et al. [5]. The authors found that cracks should be incorporated in simulations for correctly estimating  $t_i$  in RC members exposed to chlorides. Yu et al. [6] conducted an experimental investigation and the results revealed that exposure conditions and cover depth influence corrosion evolution over time in cracked RC beams exposed to chlorides. The experimental campaign of Otieno et al. [7][8] on naturally and artificially corroded beams investigated corrosion propagation in marine environments and

led to the development of two corrosion rate models. The study revealed that laboratory tests employing accelerated corrosion techniques cannot be used to infer insights on the field performance under natural corrosion. The influence of crack width, cover depth, binder type and water to binder ratio  $w/b$  on corrosion rate have also been investigated, indicating that corrosion rate increases as crack width increases and decreases as concrete quality and cover depth increase. It has been pointed out that the reported corrosion rates were averaged over the whole steel surface, but the assumption leads to underestimating corrosion rates and penetration depth. In cracked concrete this is particularly important since high local pitting is expected in proximity of the cracks. Also, even if the proposed corrosion rate models match well the experimental data, independent data sets are needed to assess the accuracy of the models. The work of Kathler et al. [9] confirms the general agreement on the fact that concrete cracking is associated with accelerated chloride penetration and, consequently, corrosion initiation but highlights that data regarding the influence of cracks on corrosion propagation are contradictory, possibly because of different test and exposure conditions and lack of time-resolved data in cyclic-wetting exposure. The influence of cyclic wet-dry exposure on cracked concrete has been investigated experimentally and a corrosion rate model characterized by exponential decay over time has been developed. Jones et al. [10] simulated the effects of crack-filling materials used to repair cracked concrete and delay corrosion initiation and found that cracks allow chloride to bypass the bulk concrete significantly reducing the effectiveness of concrete cover depth. They also pointed out that the effectiveness of repair strategies is strictly related to their effect on the diffusivity of the damaged zone.

Leung and Hou [11] carried out extensive numerical research resulting in the empirical equation of an equivalent diffusion coefficient  $D_{eq}$  accounting for concrete cracking.  $D_{eq}$  implicitly includes diffidence effects in the crack and has been found to be accurate for typical concrete cover depth of RC members. The study of Otieno et al. [12], based on an experimental campaign on cracked beam specimens, introduced the concept of performance-based crack width limits, since a universal value is not valid for a wide range of exposure conditions and cover depths. The limits should take into account the complexity of the interaction between concrete quality, cover depth, crack characteristics and exposure conditions.

Bezuidenhout and van Zijl [13] studied experimentally the influence of multiple cracks and reinforcement layouts on the corrosion process. The authors also proposed a model to estimate the residual service life of a structure that entered the propagation phase. The work of Russo et al. [14] on corrosion of carbon and stainless-steel bars confirmed the discrepancies in data regarding corrosion propagation in cracked concrete. The results highlighted that micro-cracks severely accelerate corrosion on carbon steel rebars, while stainless-steel reinforcement was still in passive condition even in cracked concrete. O'Reilly et al. [15] highlighted that even narrow cracks (0.025 to 0.1 mm) can lead to

sensible reduction of  $t_i$ . Zhang et al. [16] investigated how environmental humidity, load-induced crack and stress state affect the chloride diffusion process in concrete, studying corrosion initiation through a probabilistic approach.

Given that RC structures crack under service loads, current models used for chloride propagation modeling are less robust than expected. Specializing the problem to Life Cycle seismic assessment of RC bridges, the interaction between corrosion and seismic performance is traditionally considered only in terms of evolution of performance metrics and decision variables over time on deteriorating structures. Serviceability level earthquakes may induce cracking on bridge piers that, if not promptly repaired, can affect corrosion initiation and significantly reduce structural integrity. It is worth noting that cracks induced by corrosion and earthquakes are different. In fact, the former are typically parallel to bar alignment and are due to expansion of corrosion products, while the latter are perpendicular to longitudinal bars, as depicted in Figure 5.1.

The goal of this chapter is to develop a practical methodology to study the mutual interaction between environmental and seismic hazard and its implications on durability and Life Cycle seismic assessment of RC bridges. The model used for taking into account cracking in the chloride penetration phase is first introduced and validated against experimental data. Given that experimental data on chloride diffusion in cracked concrete are characterized by relevant dispersion and are heterogenous in terms of crack creation methods, concrete mixes and exposure conditions, the model proposed by Leung and Hou [11] has been used to provide a transparent and reproducible methodology. The concept of the methodology used to incorporate earthquake-induced cracking in the chloride diffusion problem is then presented, followed by an illustrative example. The implications on corrosion propagation and corrosion-induced damage are discussed through a probabilistic approach. Finally, the implications on Life Cycle seismic assessment of RC bridges are investigated through the application of Performance Based Earthquake Engineering (PBEE) principles. The results are carefully scrutinized and critically discussed, highlighting the differences observed with respect to the current state-of-the-art approach.

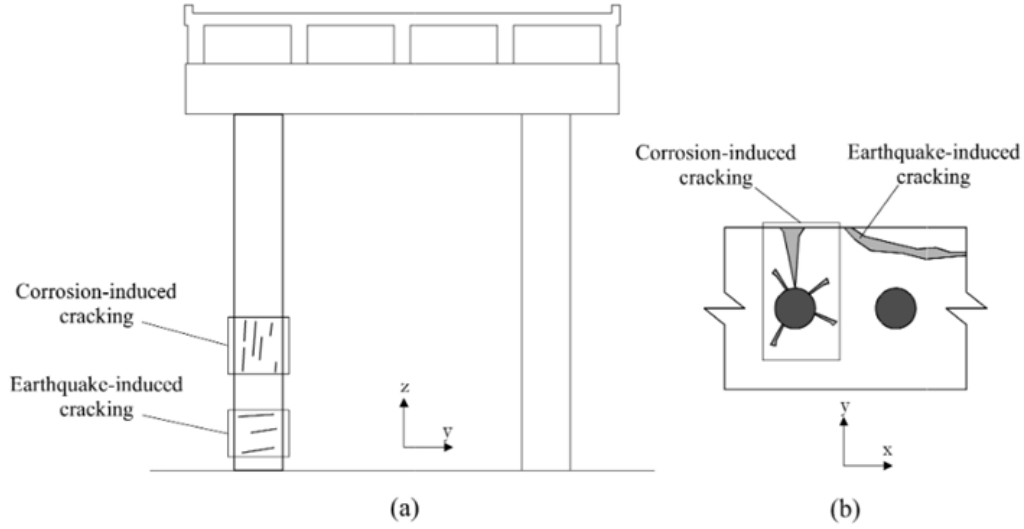


Figure 5.1. Representation of corrosion- and earthquake-induced cracking on a bridge bent. (a) elevation view, (b) detail of a typical section.

## 5.2 CHLORIDE DIFFUSION IN CRACKED CONCRETE

The model proposed by Leung and Hou [11] is used in this study to model chloride diffusion in cracked concrete. A brief description of the model is reported, and the interested reader may refer to [11] for details on its derivation. Diffusion through a crack in the concrete matrix is affected by diffiulce effects in the normal direction of the crack. The equivalent diffusion coefficient  $D_{eq}$  can be defined as per Equation (5.1):

$$(5.1) \quad D_{eq} = \frac{D_{cr} + D_0 \cdot k \cdot \left(\frac{D_0 \cdot t}{x^2}\right)^b}{1 + k \cdot \left(\frac{D_0 \cdot t}{x^2}\right)^b}$$

Where  $b$  and  $k$  are parameters expressed as per Equations (5.2)-(5.3):

$$(5.2) \quad b = \frac{\left(\frac{D_{cr}}{D_0}\right)^{0.201}}{0.107 \cdot \left(\frac{D_{cr}}{D_0}\right)^{0.484} + 5.973 \cdot \left(\frac{x}{w}\right)^{-0.109}}$$

$$(5.3) \quad k = \frac{\left(\frac{x}{w}\right)^{0.973}}{0.390 \cdot \left(\frac{D_{cr}}{D_0}\right)^{-0.0388} - 3.024 \cdot 10^{-8} \cdot \left(\frac{x}{w}\right)^{1.755}}$$

In the previous equations,  $D_{cr}$  is the chloride diffusivity of the crack,  $D_0$  is the chloride diffusivity of uncracked concrete,  $x$  is the distance from surface and  $w$  is the crack width.  $D_{eq}$  can be used to solve the 1D chloride diffusion problem to quantify the chloride

concentration at a given depth at a certain time. The model will be used in the following sections for this purpose, applying it to solve the 1D diffusion equation in the case of cracked concrete. Future developments of the present work may include a more refined model, including additional parameters related to the crack development in the concrete matrix.

### 5.2.1 Validation

Validation against experimental data of the model proposed by Leung and Hou [11] for using it in the next sections of this chapter has been carried out. Experimental data regarding chloride diffusion in cracked concrete have been retrieved from literature studies [5][17]-[22]. In order to provide meaningful comparison, specimens with different concrete composition have been included in the investigation. Crack width values up to 0.5 mm have been considered, given that most experimental investigations include data up to this point and only a little information is available for higher crack width values. Chloride diffusion coefficient values have been normalized to their value in uncracked condition. Finally, for the sake of comparison, the formulation proposed by Kwon et al. [17] for computing the average chloride diffusion coefficient in cracked concrete  $D_{avg}$  has been considered.  $D_{avg}$  can be computed as per Equation (5.1):

$$(5.4) \quad D_{avg} = (31.61w^2 + 4.73w + 1) \cdot D_0 \quad w \geq 0.1 \text{ mm}$$

The results of the comparison have been reported in Figure 5.2. It is worth noting that the model proposed by Kwon et al. [17] is valid only for  $w \geq 0.1$  mm and, for this reason, its values have been computed only in that range. It can be observed that the few available data are affected by significant scatter. Non negligible discrepancy has been observed between experimental data and analytical models, especially for  $w \geq 0.1$  mm. Multiple reasons can explain this result. First, both models are governed by  $w$ , neglecting other possible relevant parameters. While  $w$  is commonly accepted as one of the governing parameters of the problem, other factors such as crack geometry [18][22], concrete composition [20] and matrix microstructure [19][23] are certainly relevant and are not explicitly considered by both models. Second, due to the scarcity of available data, results obtained through different testing methodologies, test and exposure conditions, field and laboratory investigations have been aggregated. On one hand, this operation increases the scatter between data, integrating results with different levels of uncertainty. On the other hand, it confirms that  $w$  is not the only governing parameter of the problem. Finally, both models have been calibrated using a limited dataset, limiting their representativeness.

Due to the explorative nature of the present investigation, the model proposed by Leung and Hou [11] has been used to simulate the effect of concrete cracking on chloride diffusion. It is however necessary to point out that the comparison with experimental data

highlighted the limitations of this simplified model for capturing realistically the effects of cracking on chloride diffusion. Future refinements of this investigation will be possible as more sophisticated models and data will be available.

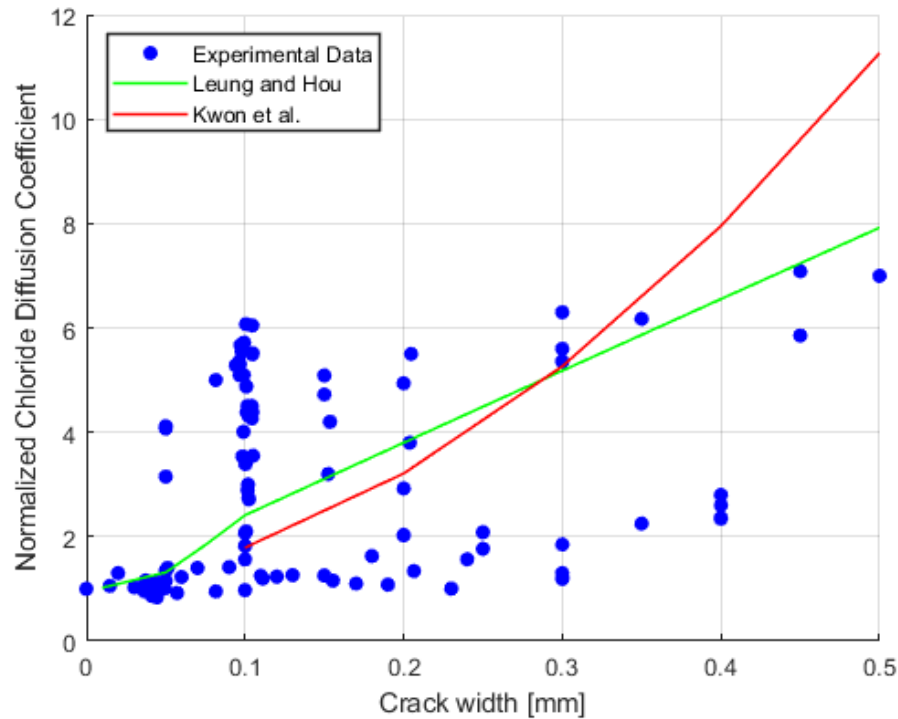


Figure 5.2. Comparison of experimental data and models for chloride diffusion coefficient in cracked concrete.

### 5.3 INTERACTION BETWEEN ENVIRONMENTAL AND SEISMIC HAZARD AND IMPLICATIONS ON CORROSION INITIATION

This section introduces and discusses the procedure used to investigate the effect of cracks induced by serviceability level earthquake on chloride diffusion and corrosion initiation. To the author's best knowledge, no similar attempt can be found in literature.

Figure 5.3 provides a general outline of the approach. The approach has been implemented through the combination of in-house MATLAB routines and the OpenSees package. The algorithm is applied with time resolution of one year for a defined time interval. Monte Carlo simulation is selected as the analysis approach. Each parameter of the 1D diffusion process is modeled as a Random Variable (RV) with statistical distribution retrieved from the literature. Each realization of the parameters is employed for the simulation of the

chloride diffusion process. For each step of the simulation, if concrete is uncracked, the occurrence of a certain level of Intensity Measure (IM) is simulated consistently with the site-specific seismic hazard curve. An equivalent static force is defined based on the sampled IM value and then the Eurocode 2 model [4] is applied to compute crack width. If concrete is cracked, the resulting crack width is used as the starting point for computing the 1D diffusion process parameters, while the regular 1D chloride diffusion simulation is performed according to [25][26] if concrete is uncracked. Finally, after completing the Monte Carlo simulation, the probability of corrosion initiation over time, for longitudinal and transverse reinforcement, is computed.

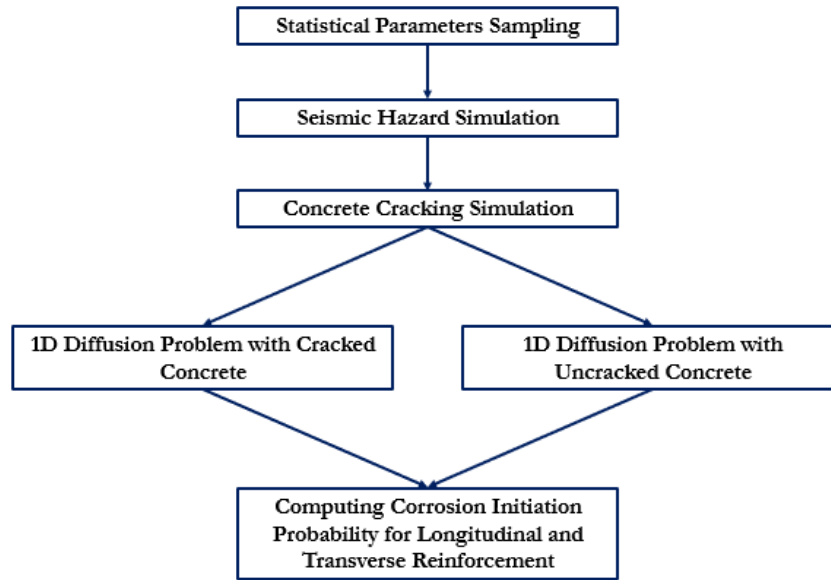


Figure 5.3. General outline of the proposed approach.

### 5.3.1 Seismic Hazard Simulation

Seismic hazard is simulated based on the site-specific hazard curve. The mean annual frequency of exceedance of every IM level  $\lambda$  is transformed into the probability of exceedance in one year  $P$  according to Equation (5.5):

$$(5.5) \quad P = 1 - e^{-\lambda T}$$

For each step of the simulation, a random number between 0 and 1 is generated. The two points of the hazard curve related to the spectral acceleration of the first transverse period of a pier characterized by probability of exceedance immediately higher and lower of the

random number are identified and the IM level corresponding to the random number is found through linear interpolation.

As a simplifying assumption, seismic action on the piers is described through an equivalent static force  $F$  defined as per Equation (5.6):

$$(5.6) \quad F = S_a(T_{1,p}) * M$$

Where  $S_a(T_{1,p})$  is the spectral acceleration corresponding to the first period of the pier and  $M$  is the tributary mass acting on the pier.  $F$  is applied to a simplified model of the pier developed using OpenSees. Note that this corresponds to assuming that actions and stresses acting on the pier are originated by a simplified representation of site seismicity (consistent with site-specific Probabilistic Seismic Hazard Analysis) and are not directly related to a specific seismic event.

### 5.3.2 Concrete Cracking Simulation

The Eurocode 2 [4] estimation model for crack width is used to simulate concrete cracking due to seismic actions. Mean crack width  $w_m$  and maximum crack spacing  $s_{r,max}$  are estimated based on Equations (5.7)-(5.8):

$$(5.7) \quad s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \frac{\emptyset}{\rho_{p,eff}}$$

$$(5.8) \quad w_m = s_{r,max} \cdot \frac{\sigma_s - k_{td} \cdot \frac{f_{ctm}}{\rho_{p,eff}} (1 + \alpha_e \cdot \rho_{p,eff})}{E_s}$$

where  $c$  is the cover concrete depth,  $k_1$  is a coefficient accounting for the bond properties,  $k_2$  is a coefficient accounting for strain distribution,  $\emptyset$  is the bar diameter,  $\rho_{p,eff}$  is the effective tension area,  $\sigma_s$  is the stress in tension reinforcement,  $k_{td}$  is a factor accounting for load duration (equal to 0.6 for short-term loads such as the serviceability level seismic action),  $\alpha_e$  is the ratio between steel elastic modulus  $E_s$  and concrete secant elastic modulus and  $f_{ctm}$  is the mean concrete tensile strength. It is worth pointing out that using this model to estimate  $w_m$  is a simplification, but it is deemed to be acceptable given that this explorative study mainly refers to serviceability level earthquakes. Given that the seismic action is simulated through a quasi-static approach, describing it as an equivalent static force, dynamic and cyclic effects (potentially leading to cracks closure) are neglected in the present investigation, but will be addressed in future refinements of this study as more refined models and data will be available.

$w_m$  is assumed to represent the mean value of a lognormal distribution, assuming a standard deviation equal to  $0.3w_m$ . The crack width value represents the starting point for modeling



the diffusion problem in a cracked concrete matrix. The chloride diffusivity in the crack  $D_{cr}$  [m<sup>2</sup>/s] is defined according to Equation (5.9), as proposed by Djerbi et al. [5]:

$$(5.9) \quad D_{cr} = \begin{cases} (0.16 \cdot w_m - 3) \cdot 10^{-10}; & 30 \mu m \leq w_m \leq 100 \mu m \\ 13 \cdot 10^{-10}; & w_m > 100 \mu m \end{cases}$$

Literature studies, such as [18], report that only cracks wider than 55  $\mu m$  significantly affect chloride ingress, while Aldea et al. [23] noted little effect of cracks with width less than 100  $\mu m$ . For this reason, the effects of cracks narrower than 50  $\mu m$  have been neglected in the present study. It is worth noting that further improvements to the present work may include self-healing effects to compute an effective  $w$  value. It is also worth noting that the present investigation only considers the influence of cracking due to serviceability level earthquakes, but splitting cracks due to the expansion of corrosion products can strongly affect the corrosion rate during the propagation phase. While this effect has been neglected in the current investigation due to its explorative nature, the complete coupling of seismic and environmental hazard should describe the influence of splitting cracks on the corrosion rate using ad-hoc models such as the one proposed in [24]. Finally, it has to be pointed out that no experimental data are currently available to validate the present approach, representing a research gap that needs to be addressed in the future in order to critically assess the results and compare the relevance of cracking due to seismic events to other important factors in corrosion development such as weather conditions and soil humidity.

### 5.3.3 1D Diffusion Problem

According to the *fib* model [25][26], the error function solution of 1D Fick's Second Law of diffusion is used to estimate the time- and depth-dependent chloride concentration  $C(x,t)$  in concrete:

$$(5.10) \quad C(x,t) = C_0 + (C_{s,\Delta x} - C_0) \cdot \left[ 1 - \operatorname{erf} \left( \frac{x - \Delta x}{2 \cdot \sqrt{D_{app}(t) \cdot t}} \right) \right]$$

In Equation (5.10),  $C_0$  is the initial chloride content,  $C_{s,\Delta x}$  is the chloride content resulting from environmental exposure at depth  $\Delta x$ ,  $\Delta x$  is the depth of the convection zone,  $C(x,t)$  is the chloride content at depth  $x$  at the time  $t$ ,  $\operatorname{erf}$  is the Gaussian error function and  $D_{app}(t)$  is the apparent chloride diffusion coefficient at time  $t$ . Equation (5.10) is applied regularly in case of uncracked concrete, following [25][26], while  $D_{eq}$  is used in place of  $D_{app}(t)$  in case of cracked concrete.

### 5.3.4 Computation of Corrosion Initiation Probability

According to the common practice, chloride-induced corrosion is assumed to start when the critical concentration of free chloride ions  $C_{crit}$  on the surface of steel bars is reached. Due to the significant uncertainty in chloride diffusion modeling, a probabilistic approach is necessary. A limit state function  $L(t)$  is defined as per Equation (5.11):

$$(5.11) \quad L(t) = C_{crit} - C(x, t)$$

The probability of corrosion initiation can then be defined as per Equation (5.12):

$$(5.12) \quad P[L(t) \leq 0] = P[C_{crit} - C(x, t) \leq 0]$$

Note that corrosion initiation probability is computed for both longitudinal and transverse reinforcement, defining a dedicated limit state function for each one.

### 5.4 ILLUSTRATIVE EXAMPLE

An illustrative example of the approach is provided through the application on a prototype bridge structure (identified as Bridge 2 in Table 1.1 and Table 1.2). The structure is located in Reggio Calabria (Italy), which is a site characterized by high seismic hazard, and it has been designed assuming a Nominal Life of 100 years. It has been assumed that the environmental exposure class was compatible with XS3 (according to the Eurocode classification), corresponding to splash and spray zones in marine environments. The bridge consists of two simply supported spans, each 30 m long. The deck is composed of six PC beams with a 0.25 m thick upper slab. The bent includes three columns with square sections with a connecting cap beam. The columns have 0.9 x 0.9 plan dimensions and are 8 m high. 36  $\Phi 24$  bars have been used as longitudinal reinforcement of the sections, while four legs  $\Phi 10$  stirrups with 150 mm spacing have been used as transverse reinforcement. The nominal concrete cover  $c_{nom}$  to longitudinal reinforcement was taken as 60 mm, according to the Eurocode prescriptions. A  $w/c$  ratio of 0.45 has been assumed. A sketch of the case study bridge is showed in Figure 5.4.

The seismic hazard curve (Figure 5.5) has been computed using REASSESS [27] assuming a  $V_{s,30}$  value of 300 m/s. Table 5.1 reports a summary of the RVs included in the simulation [25][26].

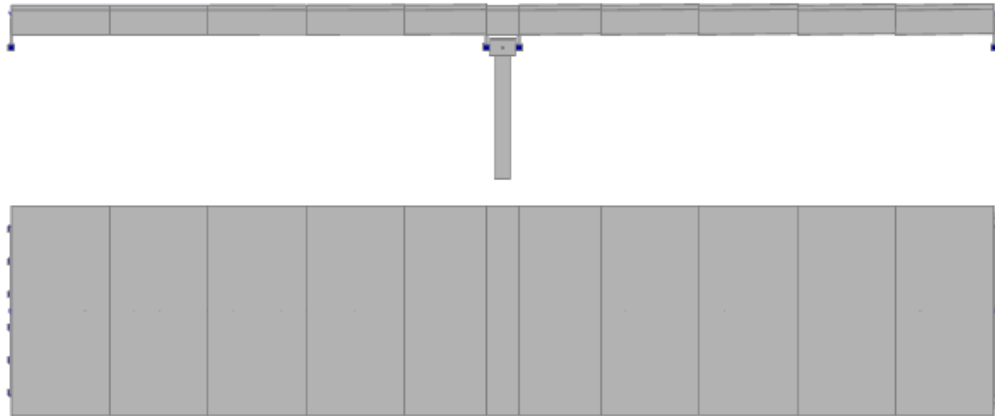


Figure 5.4. Sketch of the prototype bridge.

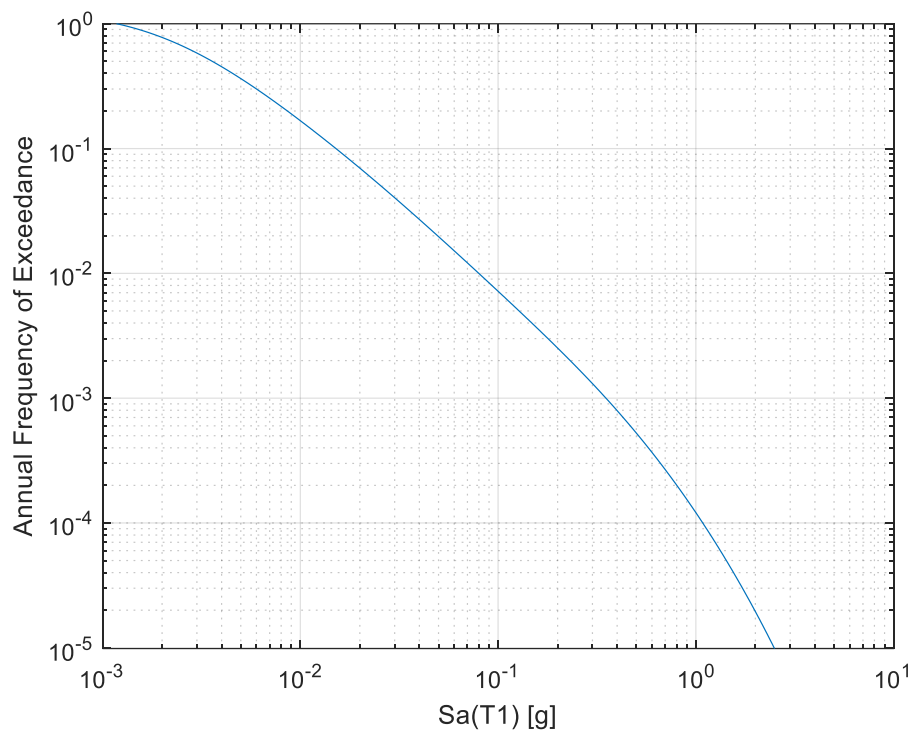


Figure 5.5. Site specific Seismic Hazard curve for Reggio Calabria.

Table 5.1. Characteristics of the RVs considered in the example application, retrieved from [25][26].

| RV                                         | Distribution Type | Mean ( $\mu$ ) | Standard Deviation | Bounds                     |
|--------------------------------------------|-------------------|----------------|--------------------|----------------------------|
| $D_{rm,0}$ [ $10^{-12}$ m/s <sup>2</sup> ] | Normal            | 4.5            | $0.2\mu$           | $\geq 0$                   |
| $a$ [-]                                    | Normal            | 0.4            | 0.16               | $0 \leq a \leq 1$          |
| $T_{real}$ [K]                             | Normal            | 292            | 5                  | -                          |
| $b$                                        | Normal            | 4800           | 700                | -                          |
| $C_{s,\Delta x}$ [wt-%/c]                  | Lognormal         | 4              | 1.8                | $\geq 0$                   |
| $\Delta x$ [mm]                            | Beta              | 10             | 5                  | $0 \leq \Delta x \leq c-1$ |
| $C_{crit}$ [wt-%-c]                        | Beta              | 0.6            | 0.15               | $0.2 \leq C_{crit} \leq 2$ |
| $c$ [mm]                                   | Normal            | $c_{nom}$      | 6                  | $\geq \varphi t + 1$       |
| $f_c$ [MPa]                                | Lognormal         | 43             | 5                  | $\geq 0$                   |
| $f_y$ [MPa]                                | Lognormal         | 495            | 30                 | $\geq 0$                   |

#### 5.4.1 Effects of Concrete Cracking on Corrosion Initiation

The effects of concrete cracking on corrosion initiation are investigated by comparing the corrosion initiation probability over time obtained by applying the proposed approach and those obtained under the regular assumption of uncracked concrete. The results have been computed for a 200-year time window, including 25000 realizations in the Monte Carlo simulation. The simplified numerical model of the pier, accounting for uncertainty in concrete and steel mechanical properties, has been developed using OpenSees [28]. Nonlinear Force-Based elements using the modified Gauss-Radau integration method [29] have been used for the bent columns, while the pier cap has been modeled using elastic beam elements. The length of the plastic hinge is estimated according to Priestley et al. [30]. The equivalent static force used to simulate the seismic forces is applied in correspondence of the geometric center of the deck, located at the height of 9.7 m.

Corrosion initiation probability for longitudinal and transverse reinforcement, including and not including possible concrete cracking due to serviceability level earthquakes, is represented in Figure 5.6. Clearly, the probability of corrosion initiation is higher for transverse reinforcement than for longitudinal rebars due to the lower depth of cover concrete. Comparing the case characterized by uncracked concrete assumption with the case including concrete cracking, it is evident that the latter is characterized by higher corrosion initiation probability over the entire time window. Further comparison is provided by following the approach introduced in [31], identifying as First Repair Time the year corresponding to the median of the cumulative distribution of corrosion initiation probability for longitudinal reinforcement. Physically, the First Repair Time corresponds to the estimated year when the first repair due to corrosion of the structure will be needed.

The quantity has been computed for both the analysis cases and absolute time difference  $\Delta t$  and percentual reduction of First Repair Time have been computed. The results are reported in Table 5.2. For the considered scenario,  $\Delta t$  is equal to 82 years, representing a 46.33% reduction of First Repair Time, indicating significant discrepancy between the approaches.

Table 5.2. First Repair Time considering and not considering earthquake induced cracks.

| Repair Time – Cracked [years] | Repair Time – Uncracked [years] | $\Delta t$ | First Repair Time Reduction |
|-------------------------------|---------------------------------|------------|-----------------------------|
| 95                            | 177                             | 82         | 46.33 %                     |

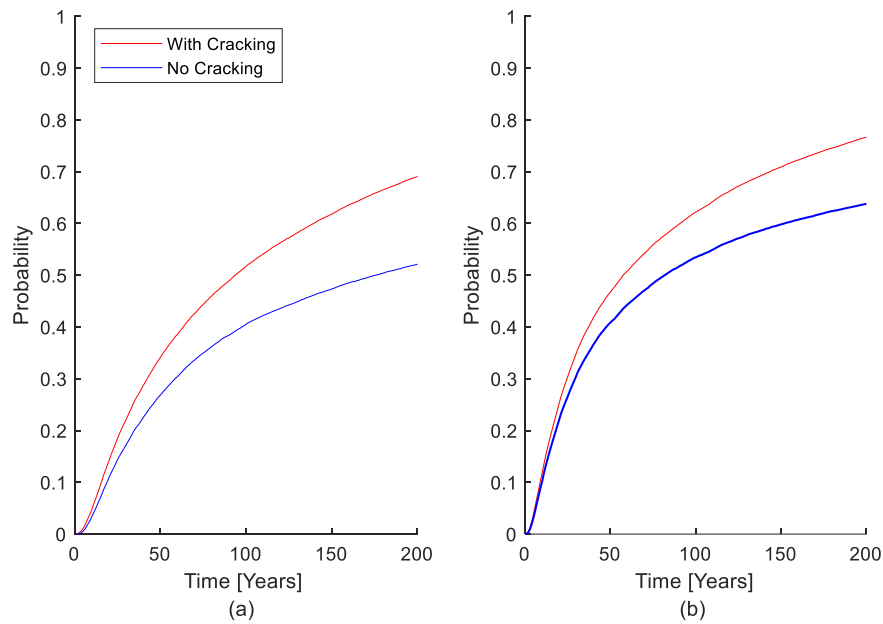


Figure 5.6. Corrosion Initiation Probability for (a) longitudinal and (b) transverse reinforcement, including and not including concrete cracking due to serviceability level earthquakes.

## 5.5 EFFECTS ON CORROSION PROPAGATION AND CORROSION-INDUCED DAMAGE

### 5.5.1 Corrosion Propagation

The implications of shortened corrosion initiation time due to earthquake-induced concrete cracking are taken into account probabilistically in the simulation. The pitting factor  $R$  is sampled from a Gumbel distribution following the indications of [32]. The reference value

of corrosion current density  $i_{corr}$  is sampled from a uniform distribution and its evolution over time is modeled as in [33]. The corrosion propagation phase can be described using Faraday's law [34]:

$$(5.13) \quad p(t_p) = 0.0116 \cdot R \cdot t_p \cdot i_{corr}$$

In Equation (5.13),  $t_p$  is the propagation period. The factor 0.0116 is present to convert  $\mu A/cm^2$  into mm/year. The corrosion current density  $i_{corr}(1)$  at the start of the corrosion propagation can be estimated empirically as per Equation (5.14):

$$(5.14) \quad i_{corr}(1) = \frac{37.8 \cdot \left(1 - \frac{w}{c}\right)^{-1.64}}{x} [\mu A/cm^2]$$

where  $w/c$  is the water-cement ratio of concrete. The relation between  $i_{corr}$  and  $t_p$  can be expressed empirically as per Equation (5.15):

$$(5.15) \quad i_{corr}(t_p) = i_{corr}(1) \cdot 0.85 \cdot t_p^{-0.29}$$

Due to the significative discrepancies that are present in the literature, the effects of cracks on  $i_{corr}$  have not been included in the simulation, indicating the need for future research. Given that the pitting corrosion model [35] is more realistic in case of chloride-induced corrosion and cracked concrete, corrosion-induced damage analysis has been carried out considering this steel area loss model. Assuming hemispherical pit morphology [36][37], the residual section in presence of pitting  $A_{R,P}$  and the pit width  $a(t)$  can be estimated through application of the model. The interested reader may refer to [35] for a comprehensive description of the model. The residual area in a given pit location  $A_{C,P}$  can be estimated according to Equation (5.16) [35]:

$$(5.16) \quad A_{C,P} = \left[1 - \frac{a(t)}{2D_0}\right] [A_{C,U}(t) - A_0] + A_{R,P}(t)$$

where  $A_0$  is the uncorroded bar area and  $A_{C,U}$  is the corroded bar area computed through the uniform corrosion model.

### 5.5.2 Time Dependent Material Properties

The formulations proposed by Du et al. [38][39] have been used to compute the corroded mechanical properties of steel material (namely yield strength  $f_y$ , ultimate strength  $f_u$  and ultimate strain  $\varepsilon_{su}$ ), based on the section loss of corroded reinforcement  $\varphi$ . The effects of cracking on cover concrete are described by reducing the compressive strength of cover concrete according to the model proposed by Coronelli and Gambarova [40], based on [41][42]. The extent of microcracks can be estimated according to [43]. Corrosion of transverse steel leads to the reduction of both ultimate compressive strength and ultimate strain of confined concrete [44], which are included through the model proposed in [44].  $\varphi$

values over time, computed including and not including concrete cracking, are represented in Figure 5.7 and Figure 5.8. The former condition is associated with more pronounced deterioration and transverse reinforcement, being characterized by smaller diameter bars and lower concrete cover depth, is more vulnerable than longitudinal reinforcement. Figure 5.9 shows the values of  $\varepsilon_{st}$  for longitudinal bars, considering and not considering cracking. The results show that material properties deterioration is naturally more pronounced if concrete cracking is included in the simulation, due to the higher value of  $\varphi$  over time.

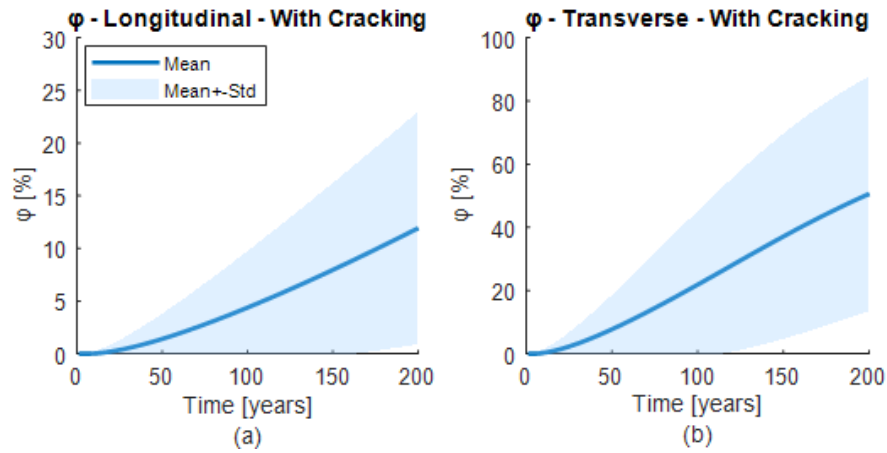


Figure 5.7.  $\varphi$  over time including concrete cracking for (a) longitudinal and (b) transverse steel.

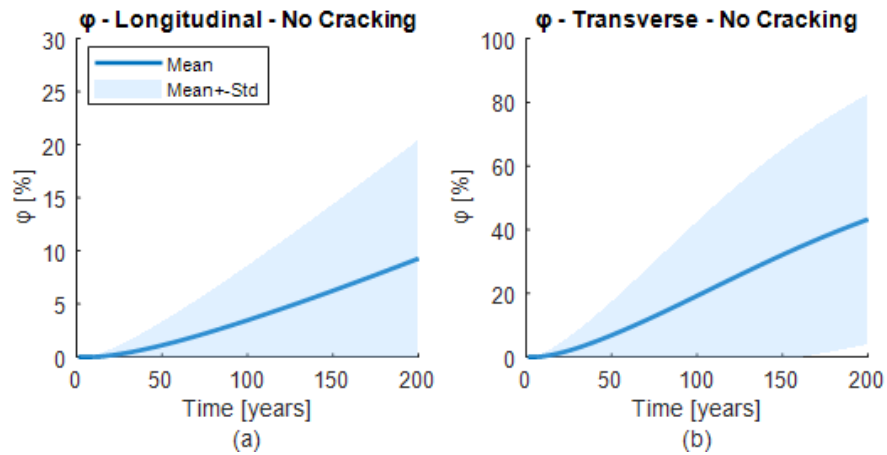


Figure 5.8.  $\varphi$  over time not including concrete cracking for (a) longitudinal and (b) transverse steel.

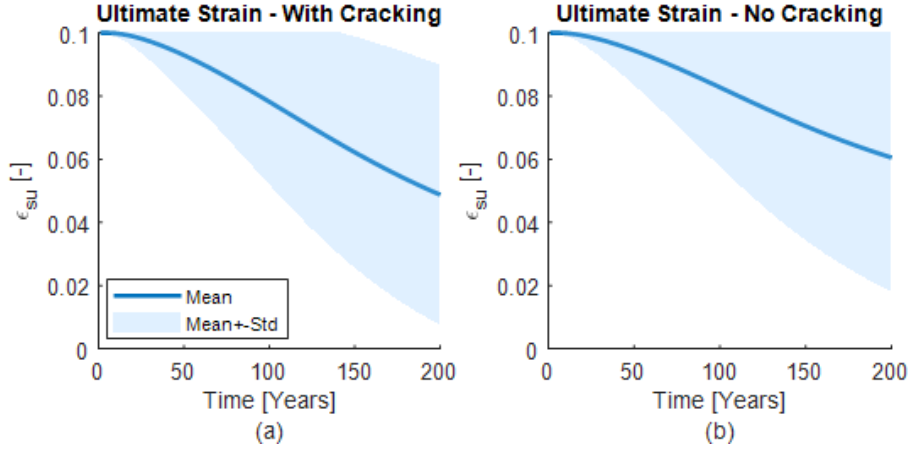


Figure 5.9.  $\epsilon_{su}$  of longitudinal bars over time, (a) including and (b) not including concrete cracking.

### 5.5.3 Corrosion-induced Damage

Corrosion-induced damage has been simulated by defining four different Corrosion-Induced Damage States (CIDS) with increasing severity, described in Table 5.3. It is worth noting that CIDS refer only to damage (such as cracking and spalling) induced by corrosion along the service life of the bridge, and each CIDS has been represented in Figure 5.10 for the sake of clarity.

The time to corrosion-induced cracking initiation  $t_{cr}$  is estimated as in [45], based on the general form of Equation (5.17), proposed in [46]:

$$(5.17) \quad t_{cr} = t_i + \left( \frac{p_{crit}}{0.0139 \cdot R \cdot i_{corr}(0)} \right)^{0.41}$$

In Equation (5.17),  $p_{crit}$  is the critical pit depth computed as the product of the amount of corrosion penetration that triggers cracking in concrete  $x_0$  (assuming uniform corrosion) and  $R$ .  $x_0$  is taken as the mean value of a lognormal distribution with constant standard deviation of 0.18.  $x_0$  can be computed according to Equation (5.18), reported in [48]:

$$(5.18) \quad \log\left(\frac{c}{D_0}\right) = \log(a) + b \cdot \log(x_0)$$

In Equation (5.18),  $a$  is a constant equal to 100.93 and  $b$  is a constant equal to 1.06. According to the literature [45][47], Equation (5.17) represents the mean  $\mu$  of a lognormal distribution whose standard deviation is equal to  $0.53\mu$ .



Corrosion-induced crack width  $w(t)$  and reinforcement area loss can be correlated according to Equation (5.19) [49]:

$$(5.19) \quad w(t) = 0.0575 \cdot |\Delta A_{i+1} - \Delta A_{CR}|$$

where  $\Delta A_{i+1} = A_0 - A_{corr}(t)$  and  $\Delta A_{CR} = A_0 - A_{CR}$ .  $A_{CR}$  is the residual steel area after cracking initiation. Note that Equation (5.19) refers to cracks with parallel orientation with respect to longitudinal bars alignment, which occur due to the expansion of corrosion products. The critical crack width for severe cracking  $w_{sev}$  is obtained from a uniform distribution with extreme values 0.2-0.3 mm [45]. Based on experimental results [50] that indicate that the value of the critical crack width for spalling  $w_{spa}$  may be well above 1 mm,  $w_{spa}$  is described as random variable with lognormal distribution characterized by logarithmic mean equal to 0.43 and logarithmic standard deviation equal to 0.39.

The curves describing the probability of exceeding the different CIDS over time including and not including earthquake-induced cracking, computed according to Equations (5.20)-(5.22), are reported in Figure 5.11. It is evident that anticipated corrosion initiation due to serviceability level earthquake-induced cracks directly affects all the damage states, which are characterized by higher probabilities with respect to the case of uncracked concrete, leading to a more probable damage on RC elements exposed to aggressive environments.

$$(5.20) \quad P[L_{crack}(t) \leq 0] = P[t_{cr} - t \leq 0]$$

$$(5.21) \quad P[L_{sev}(t) \leq 0] = P[w_{sev} - w(t) \leq 0]$$

$$(5.22) \quad P[L_{spa}(t) \leq 0] = P[w_{spa} - w(t) \leq 0]$$

To further compare the two conditions, a synthetic Corrosion Damage Index (CDI) is computed. The CDI is expressed as per Equation (5.23):

$$(5.23) \quad CDI = \sum_{i=1}^4 c_{W,i} \cdot P(CIDS = CIDS_i); CDI \in [0, 1]$$

In Equation (5.23),  $c_{W,i}$  is the damage factor for each Corrosion Damage State (assuming the values reported in Table 5.4) and  $P(CIDS=CIDS_i)$  is the probability of being in the  $i$ -th Damage State.  $P(CIDS=CIDS_i)$  can be computed as per Equation (5.24):

$$(5.24) \quad \begin{cases} P(CIDS = CIDS_i) = P(CIDS \geq CIDS_i) - P(CIDS \geq CIDS_{i+1}); \text{ for } i = 1, 2, 3 \\ P(CIDS = CIDS_i) = P(CIDS \geq CIDS_4); \text{ for } i = 4 \end{cases}$$

The values of CDI over time, considering and not considering the effect of earthquake induced cracks, are reported in Figure 5.12. It is evident that the level of damage sustained over time by the element and described by the CDI is higher if seismic action is included in the corrosion evolution simulation.

To further scrutinize the differences between corrosion damage with and without cracking effects, the probability of exceeding each Corrosion Damage State at 100 years  $P_{100}$ , which is the end of the Nominal Life of the structure, has been computed and the results have been reported in Table 5.5. The results show that including cracking effects leads to significantly higher damage probability for each Corrosion Damage State (around 27% relative difference for CIDS 1-3 and 23% for CIDS 4), with the most pronounced effects on Corrosion Initiation.

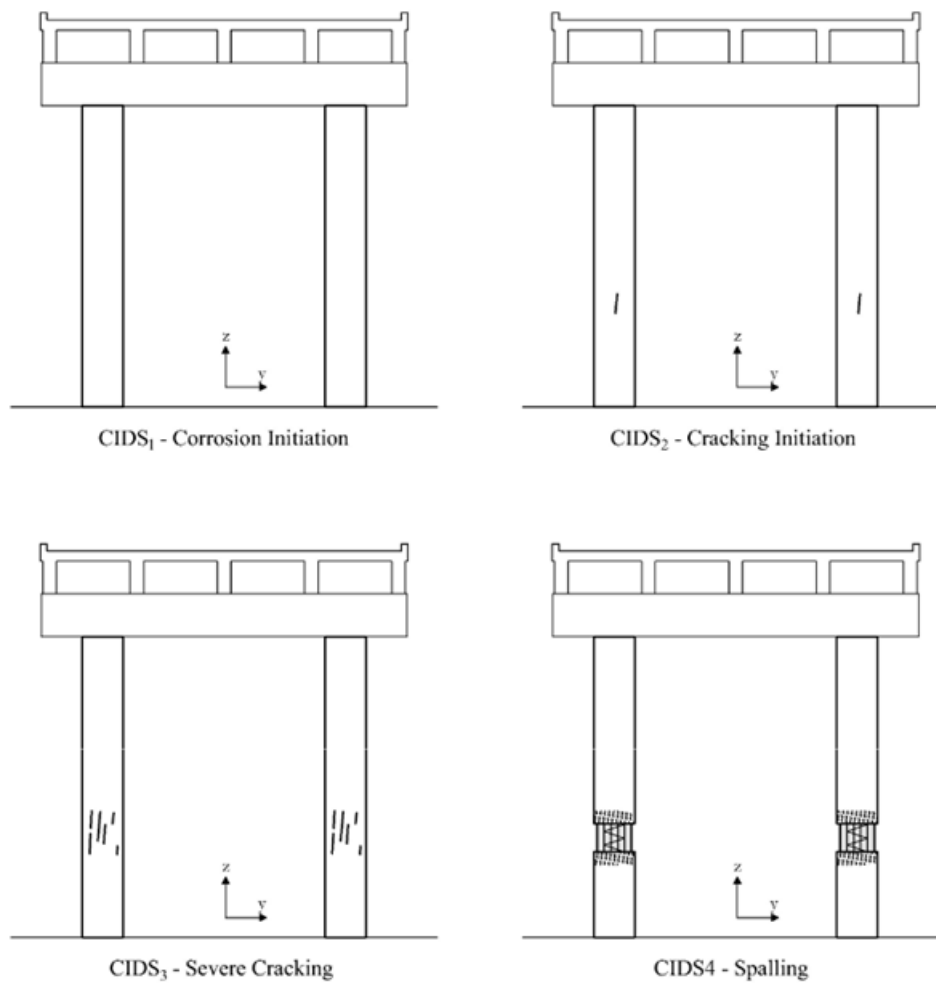


Figure 5.10. Graphical representation of each CIDS.

**Table 5.3. Corrosion Damage States**

| Damage State | Description                                          | Criteria       |
|--------------|------------------------------------------------------|----------------|
| 1            | Corrosion Initiation in Longitudinal Reinforcement   | $t=t_i$        |
| 2            | Corrosion-induced cover concrete cracking initiation | $t=t_{cr}$     |
| 3            | Severe Cracking                                      | $w(t)=w_{sev}$ |
| 4            | Spalling                                             | $w(t)=w_{spa}$ |

**Table 5.4. Damage Ratios for each Corrosion Damage State**

| Corrosion Damage State | $C_w$ |
|------------------------|-------|
| 1                      | 0.25  |
| 2                      | 0.50  |
| 3                      | 0.75  |
| 4                      | 1.00  |

**Table 5.5. Corrosion Damage State Probability at t=100 years**

| Corrosion Damage State | $P_{100}$ (with cracking) | $P_{100}$ (uncracked concrete) | Difference [%] |
|------------------------|---------------------------|--------------------------------|----------------|
| Corrosion Initiation   | 0.514                     | 0.403                          | 27.54          |
| Cracking Initiation    | 0.248                     | 0.195                          | 27.18          |
| Severe Cracking        | 0.230                     | 0.182                          | 26.37          |
| Spalling               | 0.127                     | 0.1032                         | 23.06          |

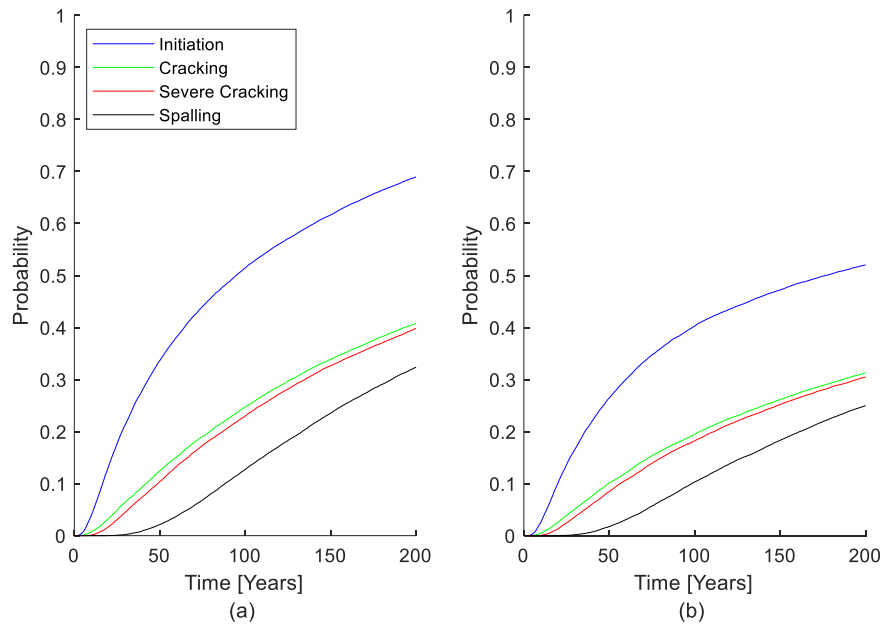


Figure 5.11. CIDS probability, (a) considering and (b) not considering earthquake induced cracking.

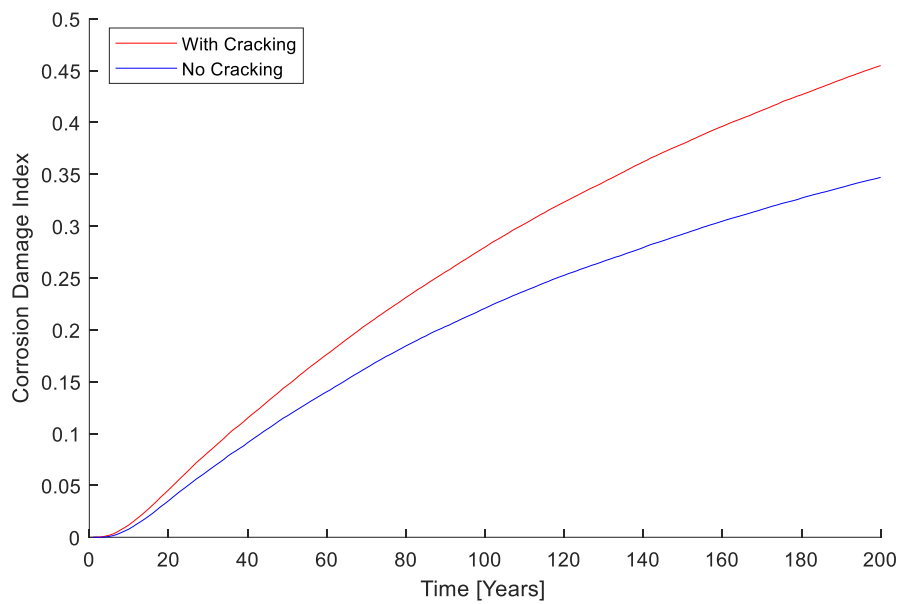


Figure 5.12. Corrosion Damage Index over time, including and not including earthquake induced cracking.

## 5.6 IMPLICATIONS ON LIFE CYCLE SEISMIC ASSESSMENT

The implications of concrete cracking induced by serviceability level earthquakes on Life Cycle Seismic Assessment are investigated in this section. It is worth noting that this should conceptually be interpreted as a Life Cycle Seismic Assessment procedure including the mutual interaction between environmental and seismic hazard, conversely to the conventional approach investigating the two parts almost independently, including the effects of deterioration on the seismic assessment but neglecting the influence of seismic cracks on deterioration evolution.

### 5.6.1 Numerical Model

The tridimensional numerical model of the prototype bridge has been developed in OpenSees [28]. The superstructure, composed of I-shaped PC beams and RC slab, has been modeled using spine elements. According to the common practice, the superstructure is assumed to respond elastically during seismic events. Five elements have been used to discretize each span of the bridge. The elastic properties of PC elements have been considered [51]. Elastomeric bearings have been modeled using *ElastomericBearingPlasticity* Elements. Bent cap elements have been modeled using elastic beams. To avoid localization problems, bent columns have been modeled with distributed plasticity elements with force-based formulation employing a modified Gauss-Radau integration method [29]. The plastic hinge length  $L_p$  was computed according to [30]. It is worth noting that  $L_p$  for corroded members has been computed using corroded model properties as inputs for the formulation reported in [30], due to the lack of experimental data for model calibration. The column sections have been defined using a fiber-based approach. *Steel02* material [52] has been used to describe the steel fibers, employing the *MinMax* material wrapper to capture bar fractures. *Concrete01* and *Concrete04* uniaxial materials have been used to model cover and core concrete fibers, respectively. The properties of confined concrete have been defined according to the model of Mander et al. [53]. P-Delta effects have been included to consider geometrical nonlinearities.

The interaction between deck and abutment backwall is modeled through *ZeroLength* elements and *ElasticPPGap* material. Material parameters have been defined according to the indications reported in [54] for seat type abutments. Material damage has been considered, avoiding recentering in presence of load reversals. The effects of foundational flexibility have been neglected, assuming that significative conservatism has been followed in the foundation design [55].

Mean material properties (computed in the corrosion propagation simulation, including and not including cracking effects) at time 0, 20, 40, 60 and 80 years have been considered in the definition of the models used for structural analysis.

### 5.6.2 Damage States

Four increasingly more severe Damage States (DS), namely Slight, Moderate, Extensive and Complete, have been considered [56]. The definition of each DS is reported in Table 5.6, while the thresholds considered for each component to compute fragility curves are reported in Table 5.7.

Table 5.6. Damage State Definition.

| Damage State | Damage Level |
|--------------|--------------|
| DS1          | Slight       |
| DS2          | Moderate     |
| DS3          | Extensive    |
| DS4          | Complete     |

Table 5.7. Thresholds for each component and for each DS.  $\Delta_y$ : yielding displacement;  $\Delta_u$ : ultimate displacement;  $d_{gap}$ : gap between deck and abutment backwall;  $d_{y,ab}$ : displacement corresponding to development of system lateral strength;  $d_{u,ab}$ : ultimate backwall displacement.

| Component | EDP               | DS1        | DS2                                   | DS3                                             | DS4        |
|-----------|-------------------|------------|---------------------------------------|-------------------------------------------------|------------|
| Piers     | Displacement      | $\Delta_y$ | $\Delta_y + 0.5(\Delta_u - \Delta_y)$ | $\Delta_y + 2/3(\Delta_u - \Delta_y)$           | $\Delta_u$ |
| Bearings  | Shear deformation | 150%       | 200%                                  | 300%                                            | Unseating  |
| Backwall  | Displacement      | $d_{gap}$  | $d_{y,ab}$                            | $\Delta_y + 2/3(\Delta_{u,ab} - \Delta_{y,ab})$ | $d_{u,ab}$ |

### 5.6.3 Seismic Hazard

Seismic response of the bridge has been characterized through Multiple Stripe Analysis (MSA), considering 10 seismic hazard levels with Return Period  $T_R$  10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 years. According to the results reported in [57][58], the 5% damped spectral acceleration corresponding to the fundamental period of the bridge  $S_d(T_1)$  was used as Intensity Measure (IM). Local soil conditions have been considered assuming a  $V_{s,30}$  value of 300 m/s. Deaggregation was performed for each IM level to characterize the events contributing to seismic hazard in terms of magnitude (M), distance to source (R) and epsilon parameter ( $\varepsilon$ ) [59]. The Conditional Spectrum approach [60][61] has been used to select hazard consistent input motions. For each IM level, 30 pairs of ground motions have been selected from the PEER database [62] matching mean and variance of the CS.

#### 5.6.4 Seismic Fragility Assessment

The conditional probability of exceeding the  $i$ -th DS is expressed through fragility functions. Component level fragility curves have been computed through the application of the Maximum Likelihood Estimation method [63] to the MSA results, estimating median IM and dispersion due to record-to-record variability for each DS. The total dispersion of each component fragility curve  $\beta_{comp}$  was computed as per Equation (5.25):

$$(5.25) \quad \beta_{comp} = \sqrt{\beta_{gm}^2 + \beta_{mod}^2 + \beta_{ls}^2}$$

Where  $\beta_{gm}$  is the dispersion component due to record-to-record variability,  $\beta_{mod}$  is the contribution of modeling uncertainty and  $\beta_{ls}$  is the uncertainty in DS definition.

$\beta_{mod}$  is computed as per FEMA P-58 [64], considering average construction (0.25) and model quality (0.25):

$$(5.26) \quad \beta_{mod} = \sqrt{\beta_c^2 + \beta_q^2}$$

In Equation (5.26),  $\beta_c$  is the modeling dispersion component due to construction quality and  $\beta_q$  is the component related to model quality.  $\beta_{ls}$  has been defined, for each component and for each DS, according to [65].

According to Nielson and DesRoches [66], the bridge system fragility functions  $P(F_{system})$  have been estimated as the upper bound of the Equation (5.27):

$$(5.27) \quad \max_{i=1}^{N_{comp}} [P(F_i)] \leq P(F_{system}) \leq 1 - \prod_{i=1}^{N_{comp}} [1 - P(F_i)]$$

Figure 5.13 shows the time dependent DS4 fragility curves, including and not including cracking effects on chloride diffusion. In both cases deterioration leads to an increase in the seismic vulnerability of the bridge, as can be observed by the progressive shift of the median value of the curves towards lower IM values. The higher corrosion initiation probability found including cracking induced by serviceability-level earthquakes in the corrosion simulation leads to higher differences over time with the results obtained with the conventional approach. In particular, the curves obtained for  $t=40$ ,  $t=60$  and  $t=80$  years are characterized by more pronounced vulnerability increase in the first case, with increasing differences as time increases. To provide a more detailed comparison, Table 5.8 reports the median values of the curves related to each DS and the relative decrease of the median value with respect to the uncorroded condition, including and not including cracking effects. It is evident that DS1 is almost constant over time. This can be explained

considering that the yielding displacement of the columns is essentially unaffected by corrosion, keeping an almost constant value over time. It can also be observed that corrosion effects are more evident for the three other DS, with increasing impact for DS3 and DS4. This can be explained considering that the ultimate displacement  $\Delta_u$  of the columns is strongly affected by corrosion of longitudinal and transverse reinforcement, with significant impact on the ductility of the concrete core and that the threshold displacements for DS2, DS3 and DS4 are functions of  $\Delta_u$ . The differences between the two approaches investigated become more relevant as simulation time increases, with differences up to around 8% in terms of relative reduction of median fragility values.

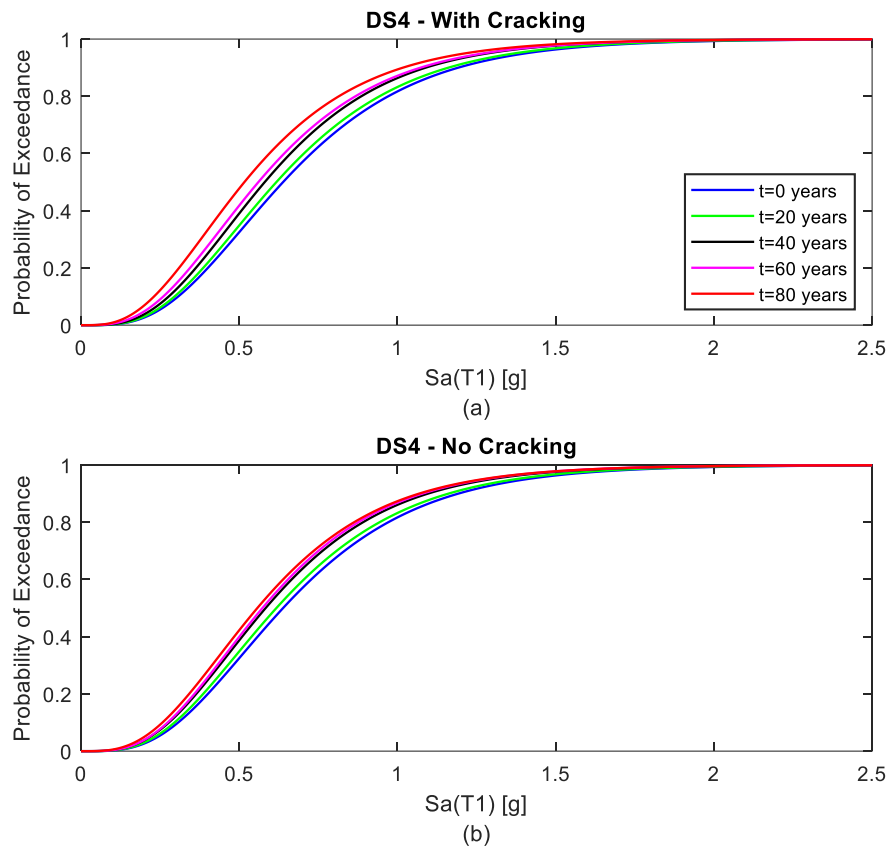


Figure 5.13. DS4 Fragility Curves – (a) including and (b) not including earthquake-induced concrete cracking.



**Table 5.8. Median IM values of Fragility Curves, with and without including concrete cracking in chloride penetration simulation. The values reported into the branches represent the percentual reduction with respect to the initial condition.**

| DS                  | Initial             | 20 years             | 40 years              | 60 years              | 80 years              |
|---------------------|---------------------|----------------------|-----------------------|-----------------------|-----------------------|
| DS1 - with cracking | 0.159 g<br>(0.00 %) | 0.159 g<br>(0.00 %)  | 0.159 g<br>(0.00 %)   | 0.159 g<br>(0.00 %)   | 0.157 g<br>(-1.19 %)  |
| DS1 – no cracking   | 0.159 g<br>(0.00 %) | 0.159 g<br>(0.00 %)  | 0.159 g<br>(0.00 %)   | 0.159 g<br>(0.00 %)   | 0.159 g<br>(0.00 %)   |
| DS2 – with cracking | 0.441 g<br>(0.00 %) | 0.431 g<br>(-2.31 %) | 0.416 g<br>(-5.69 %)  | 0.392 g<br>(-11.04 %) | 0.383 g<br>(-13.11 %) |
| DS2 – no cracking   | 0.441 g<br>(0.00 %) | 0.431 g<br>(-2.31 %) | 0.421 g<br>(-4.58 %)  | 0.402 g<br>(-8.93 %)  | 0.392 g<br>(-11.04 %) |
| DS3 – with cracking | 0.571 g<br>(0.00 %) | 0.564 g<br>(-1.17 %) | 0.532 g<br>(-6.80 %)  | 0.490 g<br>(-14.14 %) | 0.452 g<br>(-20.90 %) |
| DS3 – no cracking   | 0.571 g<br>(0.00 %) | 0.564 g<br>(-1.17 %) | 0.532 g<br>(-6.80 %)  | 0.508 g<br>(-11.05 %) | 0.484 g<br>(-15.14 %) |
| DS4 – with cracking | 0.642 g<br>(0.00 %) | 0.620 g<br>(-3.46 %) | 0.578 g<br>(-10.02 %) | 0.558 g<br>(-13.12 %) | 0.514 g<br>(-19.98 %) |
| DS4 – no cracking   | 0.642 g<br>(0.00 %) | 0.620 g<br>(-3.46 %) | 0.584 g<br>(-8.96 %)  | 0.571 g<br>(-11.06 %) | 0.558 g<br>(-13.12 %) |

### 5.6.5 Seismic Risk Assessment

Seismic Risk is computed in terms of time dependent Mean Annual Frequency of Exceedance  $\lambda_f$  (also referred to as failure rate) for each DS, which can be expressed as:

$$(5.28) \quad \lambda_f = \int_0^{\infty} P(F|IM = x) |d\lambda(IM > x)|$$

In Equation (5.28),  $P(F|IM = x)$  is the fragility function for a given DS and  $|d\lambda(IM > x)|$  is the absolute value of the derivative of the site hazard curve. Additionally, the Mean Return Period (MRP) (defined as  $\lambda_f^{-1}$ ) has been computed for each DS.

Figure 5.14 reports the time dependent  $\lambda_f$  and MRP values for each DS, for both the corrosion modeling approaches. It is clear that including cracking effects on corrosion initiation leads to higher failure rates, associated with more severe deterioration, as time passes. Note that, as for fragility curves, DS1 and DS2 are relatively insensitive to deterioration, while DS3 and DS4 are strongly affected by corrosion. Similar considerations can be expressed in terms of MRP of each DS. Significant decrease of the MRP values has been observed for DS3 and DS4, more severe when seismic cracks have been included in corrosion initiation simulation. To quantify the variation of  $\lambda_f$  and MRP over time in both conditions, time-dependent values have been normalized with respect to  $\lambda_f$  and MRP

computed at the initial time, as reported in Figure 5.15. The results revealed that  $\lambda_f$  increase and MRP decrease over time is systematically more pronounced if earthquake-induced concrete cracking is included in chloride diffusion simulation. The maximum discrepancy is observed for DS4 at 80 years, with normalized  $\lambda_f$  values of 1.62 and 1.36 including and not including concrete cracking, respectively. Considering the normalized MRP values for the same quantity, values equal to 0.61 and 0.74 are found including and not including concrete cracking, respectively.

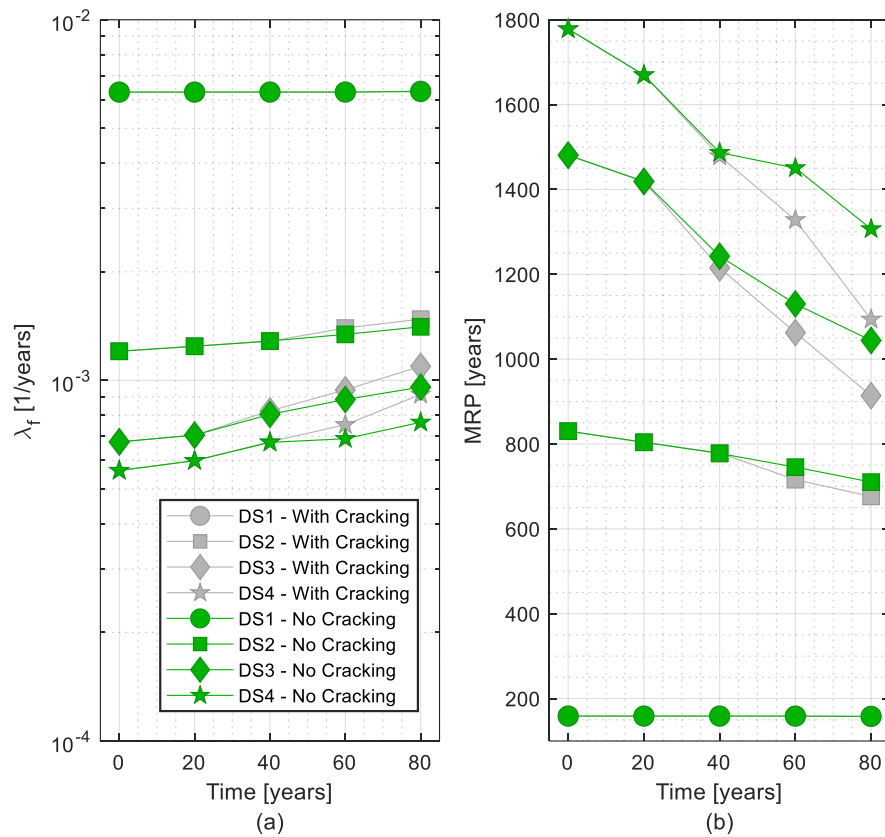


Figure 5.14.(a)  $\lambda_f$  and (b) MRP for each DS, including and not including concrete cracking.

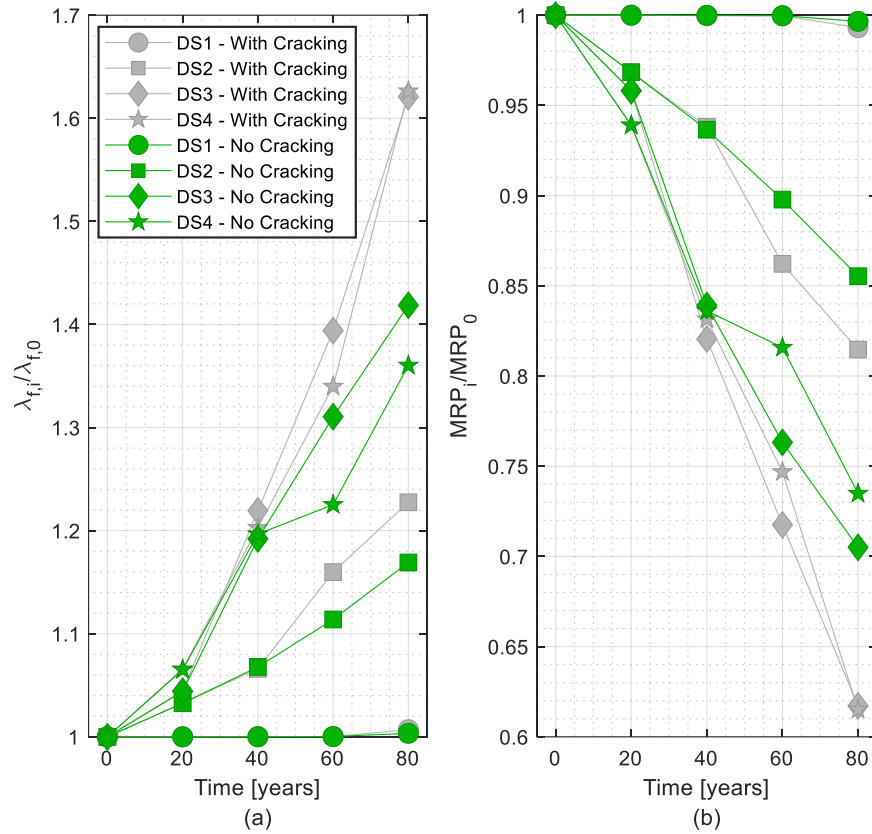


Figure 5.15. Normalized (a)  $\lambda_r$  and (b) MRP for each DS, including and not including concrete cracking.

### 5.6.6 Loss Assessment

The HAZUS approach [67] has been applied to estimate the economic loss associated with the case study bridge at each of the selected times. Intensity Based Losses, Loss Curves and Expected Annual Loss Ratios (EALR) have been computed, considering only direct losses. Time-variant Expected Annual Losses (EAL) have been used to compute the expected Life Cycle Cost (LCC) of the bridge. Table 5.9 reports the damage ratios  $\mu_{D,j}$  as per [67].

Table 5.9. Damage Ratios for each DS according to [67].

| Damage State | Damage Ratio |
|--------------|--------------|
| DS1          | 0.03         |
| DS2          | 0.08         |
| DS3          | 0.25         |
| DS4          | 1.00         |

The compound damage ratio CDR, representing the expected losses normalized on the replacement cost, is computed as per Equation (5.29):

$$(5.29) \quad CDR = \sum_{i=1}^4 \mu_{D,i} \cdot P(DS = DS_i);$$

where  $P(DS=DS_i)$  is the probability of being in the  $i$ -th Damage State.  $P(DS=DS_i)$  can be computed as per Equation (5.30):

$$(5.30) \quad \begin{cases} P(DS = DS_i) = P(DS \geq DS_i) - P(DS \geq DS_{i+1}); \text{ for } i = 1, 2, 3 \\ P(DS = DS_i) = P(DS \geq DS_4); \text{ for } i = 4 \end{cases}$$

Direct Losses can be then computed as per Equation (5.31):

$$(5.31) \quad L_D = C_{rep} \cdot CDR$$

Where  $C_{rep}$  is the replacement cost of the bridge, estimated assuming a unitary cost of 930€/m<sup>2</sup> as per [68]. For the case study bridge, a replacement cost of 669600 € has been estimated. Knowing  $CDR$ , it is possible to construct the loss curve and compute the Expected Annual Loss Ratio EALR according to Equation (5.32) [69]:

$$(5.32) \quad EALR = \int_0^\infty CDR |d\lambda(IM > x)|$$

It is worth remarking that the approach proposed by HAZUS is structure-level and generally represents an upper bound to component-level approaches, producing results that are overall well aligned with those obtained applying the latter type of approach [70]. Figure 5.16 shows the loss curves over time for the case study, considering and not considering the possible cracking effects on the chloride diffusion phase. It is clear that the mean annual frequency of exceeding a given level of expected losses increases over time. The results also show that including cracking effects in the corrosion initiation phase leads to a more pronounced increase in the mean annual frequency of a given expected loss level. Figure 5.17 displays the Intensity Based Expected Losses over time for the case study, considering and not considering the possible cracking effects on the corrosion initiation phase.

Similarly to what has been observed with the loss curve, it is possible to observe how structural deterioration leads to progressively increasing expected losses for a given IM level over time and that the increase is more pronounced if cracking effects are included. Figure 5.18 shows the time dependent EALR, including and not including cracking effects on corrosion initiation phase and the EALR values over time normalized on the initial EALR value. Progressive damage induced by corrosion propagation in the structure leads to increasing EALR values over time, with a maximum increase of around 44% and 27% including and not including possible concrete cracking. The impact of including possible concrete cracking in the initiation phase increases with time, with deviations starting for  $t=40$  years and becoming relevant for  $t=60$  years and  $t=80$  years.

Finally, LCC has been computed according to Equation (5.33):

$$(5.33) \quad LCC = \sum_{t=0}^T \frac{EAL_t}{(1+\alpha)^t}$$

where  $EAL_t$  is the EAL value at time  $t$  and  $\alpha$  is an inflation-adjusted discount rate assumed as 2%.

The LCC has been computed by investigating three conditions: deterioration evolution modeled including concrete cracking in the corrosion initiation phase, deterioration evolution modeled through the conventional corrosion development model and deterioration neglected in the assessment (assuming a constant value of EAL over time). The third condition has been used as a benchmark to investigate the importance of including deterioration in the assessment and yielded a LCC of 21438.68 €. The LCC computed including concrete cracking in the corrosion initiation phase yielded a LCC of 23849.66 €, 11.25% higher than the benchmark condition. Finally, the LCC computed through the conventional approach led to a value of 23337.07 €, 8.85% higher than the benchmark conditions. The results allow us to make some considerations. Neglecting deterioration in the LCC estimation leads to underestimating the value, which is an expected result due to neglecting the increase of EAL values over time. Comparing the approach including concrete cracking effects with the conventional one, it has been observed that the former yields slightly higher values of LCC. This result is explained by considering the fact that higher discrepancies between the EAL values of two approaches are found after a considerable amount of time has passed, reducing the impact on LCC estimation.

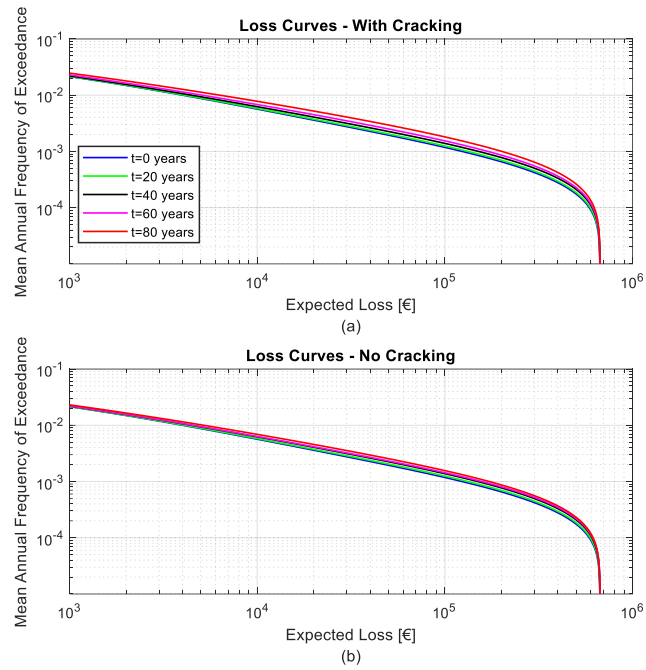


Figure 5.16. Time dependent Loss Curves, (a) including and (b) not including concrete cracking.

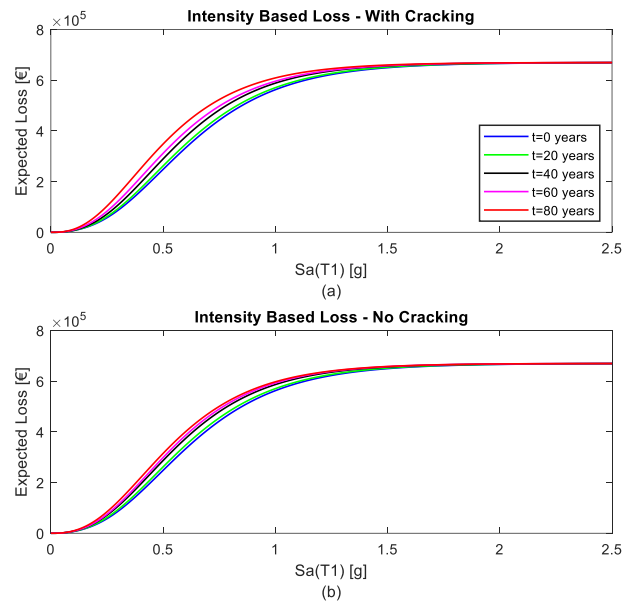


Figure 5.17. Intensity Based Expected Losses, (a) including and (b) not including concrete cracking.

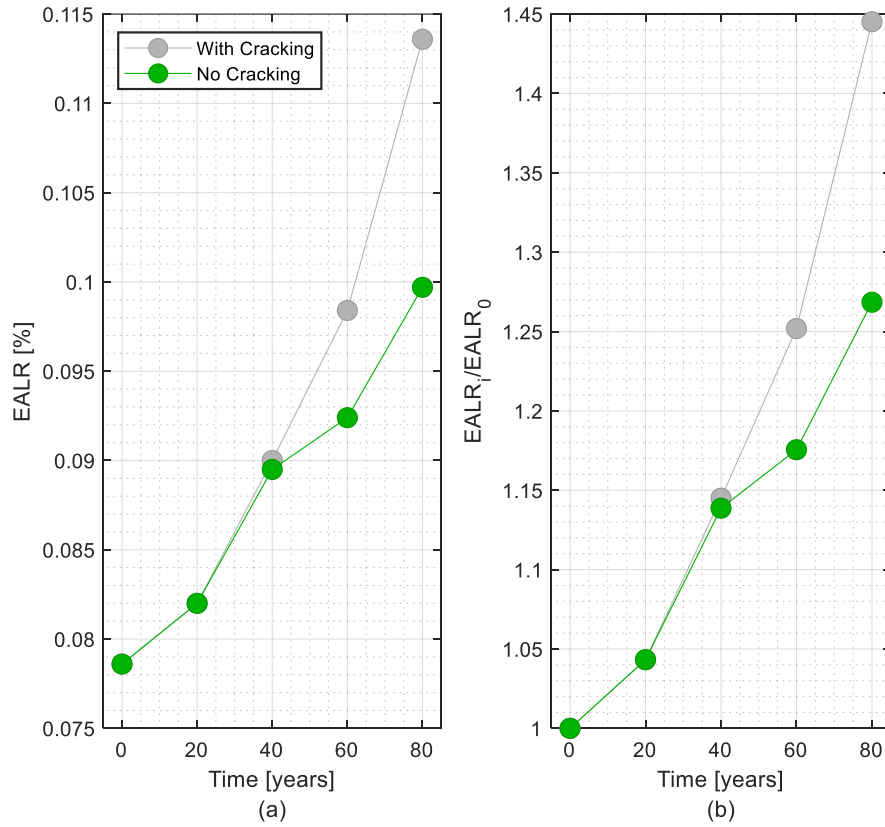


Figure 5.18. (a) EALR and (b) normalized EALR over time, including and not including concrete cracking.

## 5.7 CONCLUSIONS

This chapter extensively investigated the mutual interaction between environmental and seismic hazard and the implications on durability and Life Cycle seismic assessment of RC bridges. The probabilistic approach used in this study includes the simplified simulation of seismic hazard in the chloride diffusion simulation. Crack width is computed according to the Eurocode model. Concrete cracking is included in the corrosion initiation phase through an equivalent diffusion coefficient implicitly including diffidence effects. The effect of concrete cracking on corrosion rate has been neglected due to the contradictory results in literature. The results have been computed both including and not including earthquake-induced concrete cracking in the simulation.

Durability issues have been discussed in terms of four different CIDS: corrosion initiation, cracking initiation, severe cracking and concrete spalling. A synthetic index has been

included to summarize the results. The simulations revealed that concrete cracking significantly affects corrosion progression in RC members, with higher damage probabilities for each CIDS, highlighting how this phenomenon should be carefully considered in high seismicity regions to define appropriate maintenance and managing strategies of individual structures and networks.

The implications on Life Cycle seismic assessment of RC bridges have been investigated carrying out time dependent seismic risk and loss assessment based on the results obtained in the first portion of the study. Seismic risk has been quantified in terms of Mean Annual Frequency of Exceedance of four DS and of the corresponding MRP. The results revealed that including cracking effects in corrosion initiation phase leads to generally higher seismic risk with respect to the uncracked concrete assumption, with significant divergence over time. Seismic risk metrics have been computed based on time dependent seismic fragility curves. Seismic fragility curves revealed that corrosion increases the seismic vulnerability of the bridge over time, as demonstrated by the progressive shift of median fragility values towards lower IM levels. Economic losses have been computed in terms of loss curves, intensity based expected losses, EALR and LCC. The results revealed that deterioration leads to higher expected losses over time, with significant discrepancies for longer times. These results are confirmed also in terms of EALR over time, observing that differences up to 15% in the values can be found if cracking effects are included. The estimation of LCC has been carried out for both conditions, comparing the results with a benchmark condition neglecting deterioration. It has been found that neglecting deterioration leads to underestimating the LCC of the bridge, and that including concrete cracking in the chloride diffusion simulation leads to slightly higher values of LCC with respect to the uncracked concrete assumption.

Overall, the study revealed how in high seismicity regions, cracks induced by serviceability level earthquakes may have non negligible effects on durability and time dependent seismic performance of RC bridges. To the author's best knowledge, this is the first study addressing this topic in a systematic way. Future developments of the approach may include cracking influence on the corrosion propagation phase (even though more research is needed on the topic) and applying a component-level approach to estimate economic losses in order to provide a comprehensive investigation on the issue. Future improvements building on the present investigation may also include dynamic effects and self-healing in the crack simulation module, providing a more refined description of the interaction between serviceability-level earthquakes and chloride diffusion in cracked concrete.

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## 6. SEISMIC RESILIENCE OF DETERIORATING RC BRIDGE STRUCTURES

### 6.1 INTRODUCTION

Seismic Resilience is a key parameter for functionality assessment of structures and infrastructures following a seismic event. The meaning of this term is broad and, limiting the discussion to the field of earthquake engineering, it generally expresses the average functionality of a structure over a given time horizon. Seismic Resilience is naturally connected to several relevant facets of disaster risk mitigation, such as economic losses, network downtime, rescue delay and restoration of normal functionality after the event. It is therefore clear that a Seismic Resilience assessment framework should represent the synthesis of modern Performance-Based Earthquake Engineering (PBEE), requiring the integration of subsequent modules in order to provide reliable and theoretically consistent results. Considering the case of bridge structures located in harsh environmental conditions, it is worth noting that structural performance is not constant over time. The possible development of deterioration phenomena, such as corrosion, leads to an evolution of structural performance over time, affecting initial functionality, recovery phase and Seismic Resilience. The main goal of this chapter is defining a practical Seismic Resilience assessment framework, consistent with the principles of PBEE and specifically tailored for deteriorating bridge structures. The main theoretical concepts related to Seismic Resilience are discussed and applied to demonstrate the practicality of the framework. The effects of assumptions related to recovery time uncertainty and initial functionality are thoroughly discussed. Finally, possible future improvements to the framework are introduced, highlighting the need for additional research in order to effectively mitigate the impact of natural disasters.

#### 6.1.1 Concept of Seismic Resilience

Seismic Resilience is commonly defined as an indicator of the average functionality of structure over a given time horizon following an earthquake with a given intensity [1]. The mathematical expression of this concept is described by Equation (6.1):

$$(6.1) \quad R = \frac{1}{t_h - t_0} \int_{t_0}^{t_h} Q(t) dt$$

Where  $R$  is the resilience index, a dimensionless value in the range  $[0;1]$ ,  $Q(t)$  is the functionality of the structure, defined in the range  $[0;1]$ ,  $t_h$  is the control time (in days) which defines the extent of the time horizon and  $t_0$  is the time of occurrence of the seismic event.

The concept of Seismic Resilience is represented in Figure 6.1 (a). The occurrence of an earthquake is associated with a sudden functionality loss  $\Delta Q$ , reducing the system functionality to a residual level  $Q_r$ . After the idle time  $\Delta_i$ , which elapses before the initiation of restoration activities, functionality is recovered by following a certain recovery path, reaching the functionality target  $Q_t$  at the end of the recovery time  $t_{rec}$ .  $R$  is computed by averaging the shaded region over the defined time horizon.

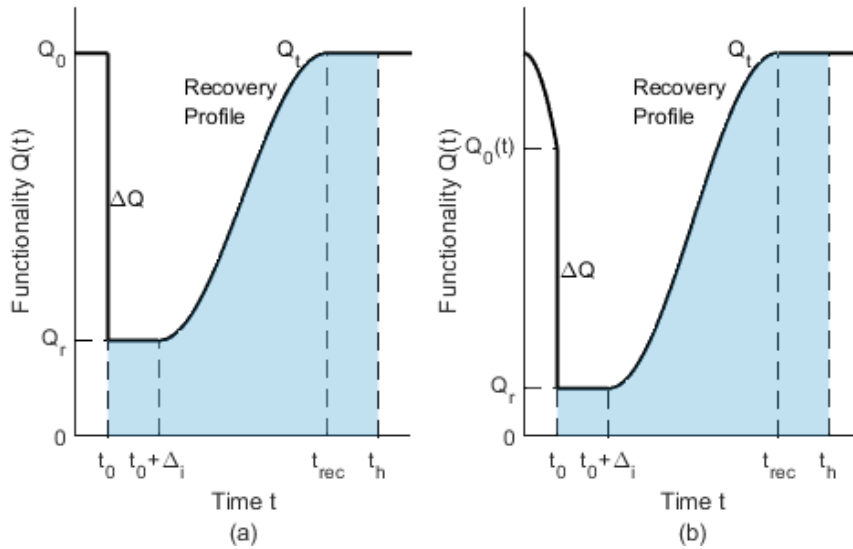


Figure 6.1. Concept of Seismic Resilience assuming (a) constant  $Q_0$  and (b) including the effects of deterioration on  $Q_0$ .

### 6.1.2 Resilience of Deteriorating Structures

The definition of Seismic Resilience introduced in the previous paragraph does not take into account the evolution of structural performance over time in the estimation of  $Q_0$  and  $\Delta Q$ . The effect of environmental stressors can lead to decreasing structural performance over time and to different levels of expected seismic losses for a given seismic event. The main consequence of this consideration is that  $Q_0$  and  $\Delta Q$  have to be assumed to depend on time  $t$ , with a number of implications on the Seismic Resilience assessment of a structure. Considering a given seismic intensity level, deterioration may lead to different levels of damage on a structure with reduced functionality, increasing both seismic losses and recovery time and, consequently, reducing the overall Seismic Resilience. The concept



of Seismic Resilience of deteriorating structures is depicted in Figure 6.1 (b). It can be observed how  $Q_0$  progressively decreases due to deterioration phenomena, including but not limited to corrosion, assuming a time-dependent dimension. Considering the functionality loss following a seismic event, structural deterioration leads to increasing  $\Delta Q$  values. While the discussion may be extended by including additional points, such as the selection of an adequate  $Q_i$  for deteriorating structures (what is the performance level that a stakeholder may want to recover after an earthquake?) and the effects of retrofit and maintenance, these facets are not included in the present investigation and the interested reader may refer to [2][3] for more insights.

### 6.1.3 Previous Research

The field of Seismic Resilience assessment has been interested by extensive research efforts over the past few years, given its inherent connection with natural hazards emergency preparedness and its applicability to critical structures and infrastructures. While a comprehensive review of the available literature is out of the scope of this investigation, this section aims at providing the fundamental insights on recent developments of the field. Biondini et al. [1] investigated the lifetime seismic performance of RC structures exposed to chloride-induced corrosion through a probabilistic approach, and found that the main factors affecting Seismic Resilience include the loss of functionality induced by an earthquake (which is time dependent due to the effects of structural deterioration), the functionality target of the recovery interventions and the recovery profile. Cimellaro et al. [4] presented a framework for the Seismic Resilience assessment of hospital systems, highlighting the possibility of using the concept of Seismic Resilience as decision support tool for critical facilities. Huang and Huang [5] proposed a framework for the Seismic Resilience assessment of aging bridges, including different failure modes. Including different failure modes may be necessary for deteriorating structures due to the possible shift of failure mode from flexural to shear-dominated in corroded RC members. The study highlights how Seismic Resilience significantly decreases as the ground motion intensity increases and that chloride-induced corrosion leads to reduction of capacity of withstanding seismic damage and rapid recovery. The study discussed in [6] proposed the combination of maximum drift ratio and residual drift ratio as indicators for the Seismic Resilience evaluation and investigated the comparison with results obtained through single indicators methods, highlighting how the double indicator method appears to be more conservative. The work of Feng et al. [7] studied the lifetime Seismic Resilience of deteriorating bridge structures under mainshock-aftershock sequences. The results highlighted how aftershocks significantly increase the fragility of bridge systems and decrease Seismic Resilience depending on a number of inter-related factors. Lu et al. [8] and Zhou et al. [9] studied the lifetime seismic performance of bridge structures including scour and corrosion effects. Both the studies agree on the fact that scour and corrosion reduce bridge functionality over time and hinder the recovery process. Corrosion appears

also to be more detrimental in terms of lifetime seismic fragility and resilience loss increase with respect to scour effects. Chirdeep et al. [10] included changing climatic conditions in the lifetime Seismic Resilience assessment and compared the results with those obtained using a conventional corrosion evolutionary model. The results underscore the significance of including climate change effects in the long-term Seismic Resilience assessment of RC infrastructures. The framework proposed in a recent work [11] investigated the role of deterioration in the recovery phase, implementing several capacity reduction rates. It is worth noting that Seismic Resilience can be used as a quantitative metric to demonstrate the efficiency of the seismic performance of specific retrofit or structural solutions. In this sense, [12] and [13] incorporated the effects on retrofit interventions in the Seismic Resilience estimation flowchart. Giouvanidis and Dong [14] investigated the possibilities offered by using rocking piers, noting significant reduction of seismic losses and increase of Seismic Resilience with respect to fixed-base configurations. Similarly, Geotechnical Seismic Isolation performance has been evaluated and compared with that of traditional seismic isolation in terms of Seismic Resilience, highlighting the advantages related to coupling both technologies [15]. The works of Zhong et al. [16] and Hong et al. [17] demonstrated the performance of novel solutions to protect bridge piers through Seismic Resilience assessment, highlighting the advantages provided in terms of loss mitigation and functionality recover.

## **6.2 SEISMIC RESILIENCE FRAMEWORK FOR DETERIORATING BRIDGE STRUCTURES**

The present section discusses the submodules of the Seismic Resilience framework for deteriorating bridge structures implemented in this work. A brief description of the corrosion evolutionary model and of the deterioration laws used for computing the time-dependent properties of a given structure is first provided. The main aspects related to seismic fragility and loss analysis are presented. Finally, the Seismic Resilience assessment approach, representing the main contribution of this chapter and oriented towards its practical implementation for the analysis of RC bridge structures, is thoroughly discussed, highlighting the key concepts of each of its components.

### **6.2.1 Corrosion Model**

The evolution over time of chloride concentration at the depth of the steel surface  $x$  is estimated according to the error function solution of 1D Fick's Second Law of diffusion [18][19]. Due to the inherent uncertainty of chloride diffusion modeling, a Monte Carlo simulation-based approach is used to estimate the time- and depth- dependent chloride concentration in RC members. It is worth noting that corrosion initiation of longitudinal and transverse reinforcement needs to be considered in order to define the deterioration laws of structural properties.

Corrosion propagation phase is described through Faraday's Law [20]. The value of the pitting factor depends on the morphology of corrosion. For the case of chloride-induced corrosion, the pitting corrosion model is considered more adequate for a realistic description of the time-dependent evolution of structures. The interested reader may refer to [21] for a description of the model. The pitting factor is modeled as a Gumbel distribution as in [22]. The reference value of corrosion rate  $i_{corr}$  is sampled from a uniform distribution and its evolution over time is modeled as in [23].

The evolution of material mechanical properties over time, according to the corrosion level, is described through deterioration models available in the literature. The evolution of steel material properties (yield strength  $f_y$ , ultimate strength  $f_u$  and ultimate strain  $\varepsilon_{su}$ ) have been computed according to the models of Du et al. [24][25]. Corrosion-induced reduction of cover concrete compressive strength has been included through the model proposed by Coronelli and Gambarova [26], based on [27]. The compressive strength  $f_c^c$  and ultimate strain  $\varepsilon_{cu}^c$  of confined concrete for corroded elements can be computed as per [30].

### 6.2.2 Seismic Hazard

10 Intensity Measure (IM) levels with return period  $T_R$  10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 are considered to select the seismic input for Multiple Stripe Analysis (MSA). According to the results reported in [31][32], the spectral acceleration at the first mode period of the bridge  $S_a(T_1)$  is used as IM. The site-specific seismic hazard curve is computed using REASSESS [33]. In order to select hazard consistent ground motions, deaggregation of the seismic hazard has to be performed at each IM level, characterizing the contributions in terms of magnitude (M), distance to source (R) and epsilon parameter ( $\varepsilon$ ) [34]. The ground motions for each IM level are then selected applying the Conditional Spectrum (CS) method [35][36] and retrieved from the PEER database [37] matching mean and variance of the CS.

### 6.2.3 Fragility Analysis

Four Damage States (DS), namely Slight (DS1), Moderate (DS2), Extensive (DS3) and Complete (DS4) are identified [38]. Thresholds corresponding to the exceedance of each DS, for each bridge component, are reported in Table 6.1. Fragility curves, expressing the conditional probability of exceeding a DS, are defined for each bridge component applying the Maximum Likelihood Estimation method proposed in [39]. The approach allows to estimate median IM and record-to-record variability dispersion component for each DS. The total dispersion  $\beta_{comp}$  is computed by considering three components as per:

$$(6.14) \quad \beta_{comp} = \sqrt{\beta_{gm}^2 + \beta_{mod}^2 + \beta_{ds}^2}$$

Where  $\beta_{gm}$  is the dispersion component due to record-to-record variability,  $\beta_{mod}$  is the contribution of modeling uncertainty and  $\beta_{ds}$  is the uncertainty in limit state definition.  $\beta_{mod}$  is computed as per the FEMA P-58 [40], considering average construction and model quality:  $\beta_{ds}$  is defined, for each component and for each DS, according to [41].

According to Nielson and DesRoches [42], system fragility functions  $P(F_{system})$  are conservatively estimated as the upper bound of the Equation (6.15):

$$(6.15) \quad \max_{i=1}^{N_{comp}} [P(F_i)] \leq P(F_{system}) \leq 1 - \prod_{i=1}^{N_{comp}} [1 - P(F_i)]$$

Where  $P(F_i)$  is the fragility function of the  $i$ -th component.

**Table 6.1. Thresholds for each component and for each DS.  $\Delta_y$ : yielding displacement;  $\Delta_u$ : ultimate displacement;  $d_{gap}$ : gap between deck and abutment backwall;  $d_{y,ab}$ : displacement corresponding to development of system lateral strength;  $d_{u,ab}$ : ultimate backwall displacement.**

| Component | EDP               | DS1        | DS2                                   | DS3                                             | DS4        |
|-----------|-------------------|------------|---------------------------------------|-------------------------------------------------|------------|
| Piers     | Displacement      | $\Delta_y$ | $\Delta_y + 0.5(\Delta_u - \Delta_y)$ | $\Delta_y + 2/3(\Delta_u - \Delta_y)$           | $\Delta_u$ |
| Bearings  | Shear deformation | 150%       | 200%                                  | 300%                                            | Unseating  |
| Backwall  | Displacement      | $d_{gap}$  | $d_{y,ab}$                            | $\Delta_y + 2/3(\Delta_{u,ab} - \Delta_{y,ab})$ | $d_{u,ab}$ |

## 6.2.4 Seismic Losses

Direct Seismic Losses are estimated according to the HAZUS approach [38]. The Damage Ratios  $\mu_{D,j}$  for each DS are reported in Table 6.2. Note that  $\mu_{D,j}$  values reported in [38] refer to the United States, but they are assumed to be valid for the scope of this investigation for consistency with previous research. Further refinement of the work may enhance this aspect in order to provide more accurate estimation of the seismic losses.

The total losses  $L$ , normalized on the replacement cost of the bridge, can be computed as:

$$(6.16) \quad L = \sum_{i=1}^4 \mu_{D,i} \cdot P(DS = DS_i);$$

where  $P(DS=DS_i)$  is the probability of being in the  $i$ -th Damage State.  $P(DS=DS_i)$  can be computed as per Equation (6.17) by considering the bridge system fragility functions:

$$(6.17) \quad \begin{cases} P(DS = DS_i) = P(DS \geq DS_i) - P(DS \geq DS_{i+1}); \text{ for } i = 1, 2, 3 \\ P(DS = DS_i) = P(DS \geq DS_4); \text{ for } i = 4 \end{cases}$$

Table 6.2. Damage Ratios for each DS according to [38].

| Damage State | Damage Ratio |
|--------------|--------------|
| DS1          | 0.03         |
| DS2          | 0.08         |
| DS3          | 0.25         |
| DS4          | 1.00         |

### 6.2.5 Damage State Identification

To identify the probable DS of the bridge at a given IM level and the consequent recovery phase, the approach proposed in [43] is implemented. The approach is based on the fragility analysis results and describes the probable DS of the bridge,  $DS_p$ , as:

$$(6.18) \quad DS_p = \begin{cases} \text{Complete if } P(DS = DS4) \geq 0.15 \\ \text{Extensive if } P(DS \geq DS3) \geq 0.25 \\ \text{Moderate if } P(DS \geq DS2) \geq 0.3 \\ \text{Slight if } (P(DS \geq DS1) \geq 0.4 \\ \text{No Damage otherwise} \end{cases}$$

### 6.2.6 Initial Functionality

The effects of corrosion-induced structural performance deterioration on the initial functionality of the structure  $Q_0(t)$  are accounted for by using the results obtained through time-dependent fragility analysis. After identifying  $DS_p$ , the value of  $Q_0(t)$  is assumed to be estimated as per Equation (6.19):

$$(6.19) \quad Q_0(t_i) = \frac{\theta_{DS_p}(t_i)}{\theta_{DS_p}(t_0)}$$

where  $\theta_{DS_p}(t_i)$  is the median of the fragility curve corresponding to  $DS_p$  at the time  $t_i$ , while  $\theta_{DS_p}(t_0)$  is the median of the same fragility curve for the bridge in pristine condition. A few comments are necessary to justify this assumption. In this study, DS exceedance is assumed to occur when normalized demand-capacity ratios exceed 1. Assuming proportionality between displacement demand and IM, the relative reduction of fragility median is used as a proxy of capacity reduction due to corrosion, which is directly considered when computing the damage thresholds associated with piers. In this sense, the ratio between  $\theta_{DS_p}(t_i)$  and  $\theta_{DS_p}(t_0)$  represents a practical and consistent proxy of  $Q_0(t)$ .

### 6.2.7 Functionality Recovery Phase

The general form of the post-event functionality recovery model is expressed as per [1]:

$$(6.20) \quad Q(\tau) = Q_r + H(\tau)f_{rec}(\tau)(Q_t - Q_r)$$

In Equation (6.20),  $Q_r$  is the post-earthquake residual functionality and is defined as  $Q_0(t) - L$ ,  $H(\cdot)$  is the Heaviside step function,  $\tau$  is a normalized time defined as  $(t - t_i) / (t_{rev} - t_i)$  and  $f_{rev}$  is the recovery function. Recovery functions are used to model bridge functionality restoration. Different restoration models, based on References [1][44][45], are used for different DS, as reported in Equations (6.21)-(6.23):

$$(6.21) \quad f_{rec}(\tau) = 1 - e^{-\alpha\tau} \text{ negative - exponential}$$

$$(6.22) \quad f_{rec}(\tau) = \frac{1 - \cos(\pi\tau)}{2} \text{ sinusoidal}$$

$$(6.23) \quad f_{rec}(\tau) = e^{-\alpha(1-\tau)} \text{ positive - exponential}$$

where  $\alpha$  is a shape parameter of the recovery functions (represented in Figure 6.2).

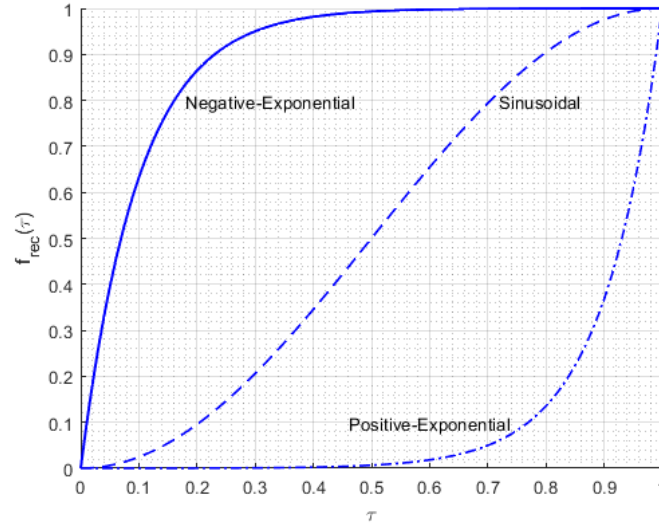


**Figure 6.2. Representation of the recovery Functions.**

The negative exponential restoration function (Equation (6.21)) represents a rapid functionality restoration, suitable for Slight DS. The sinusoidal restoration function (Equation (6.22)), used for Moderate DS, denotes a gradual restoration of functionality. Finally, the positive exponential restoration function (Equation (6.23)), denoting functionality recovery at the end of the process, is used for Extensive and Complete DS.  $t_{rev}$  for each DS is defined based on the HAZUS restoration curves for highway bridges [38][46], assumed to be applicable for the present investigation for consistency with the available literature. Further refinement of the approach may include specific  $t_{rev}$  estimation based on the context. It is worth remarking that due to the inherent uncertainty of the recovery process  $\alpha$ ,  $\Delta I_i$  and  $t_{rev}$  are characterized as RVs with distribution and parameters specified in Table 6.3.

**Table 6.3. Statistical Distributions of recovery parameters.  $\mu$ =mean;  $\sigma$ =standard deviation.**

| Parameter                | Distribution | Slight DS                    | Moderate DS              | Extensive DS                 | Complete DS             |
|--------------------------|--------------|------------------------------|--------------------------|------------------------------|-------------------------|
| $\alpha$ [44]            | Uniform      | [8-10]                       | -                        | [6-8]                        | [8-10]                  |
| $t_{rec}$ [46] [days]    | Normal       | $\mu=1.2$ ,<br>$\sigma=0.35$ | $\mu=5.2$ , $\sigma=1.9$ | $\mu=117$ ,<br>$\sigma=24.2$ | $\mu=340$ , $\sigma=63$ |
| $\Delta t_i$ [47] [days] | Uniform      | [30-60]                      | [30-60]                  | [30-60]                      | [30-60]                 |

### 6.2.8 Seismic Resilience Index

Once that the restoration curve has been obtained,  $R$  can be finally computed as per Equation (6.24):

$$(6.24) \quad R = \frac{1}{t_h - t_0} \int_{t_0}^{t_h} Q(t) dt$$

It is worth pointing out that  $t_b$  is taken as 170 days for Slight, Moderate and Extensive DS, while it is taken as 500 days for Complete DS [38]. Finally, given the probabilistic characterization of the recovery phase,  $R$  can be expressed in statistical terms considering mean value and relevant percentiles, identifying the variability of the parameter and providing further support for the rational structural management.

## 6.3 APPLICATION TO A MODERN BRIDGE STRUCTURE

The first application of the methodology is presented by considering the prototype bridge analyzed in Chapter 5 (Bridge 2 in Table 1.1 and Table 1.2), where the interested reader may find the details about case study characteristics, environmental conditions, numerical modeling and corrosion damage. The results, which refer to the conventional corrosion model not including concrete cracking, are scrutinized in order to investigate the time-variant seismic performance in terms of post-earthquake recovery and Seismic Resilience, highlighting the different recovery paths for selected IM levels for different corrosion levels and thoroughly discussing the outputs of the Seismic Resilience assessment framework.

### 6.3.1 Time-Variant Fragility Curves

Time-variant fragility curves have been computed for each DS. Figure 6.3 displays time-variant fragility curves for Complete DS. Corrosion leads to increasing seismic fragility over time, as expressed by the progressive shift of fragility curves towards lower IM levels. This effect is of capital importance in Seismic Resilience assessment, given that on one hand the level of expected losses for a specific IM level is defined based on fragility data and on the other hand deterioration is associated with a progressive reduction of pre-event functionality. Quantitative comparison between time-dependent fragility curves for each DS is provided by Table 6.4. DS1 median IM value is almost constant over time, due to

the low influence of corrosion on the yielding displacement of columns, taken as threshold for the exceedance of the DS. Corrosion effects are more relevant for the three other DS, particularly for DS3 and DS4. This effect can be explained by considering that the ultimate displacement  $\Delta_u$  of the columns is strongly affected by corrosion of longitudinal and transverse reinforcement, significantly affecting the ductility of the concrete core and that the DS exceedance thresholds for DS2, DS3 and DS4 are functions of  $\Delta_u$ . The impact of corrosion becomes more relevant on the structure as time passes, with reduction of the median values of fragility curves with respect to the uncorroded condition up to 15%.

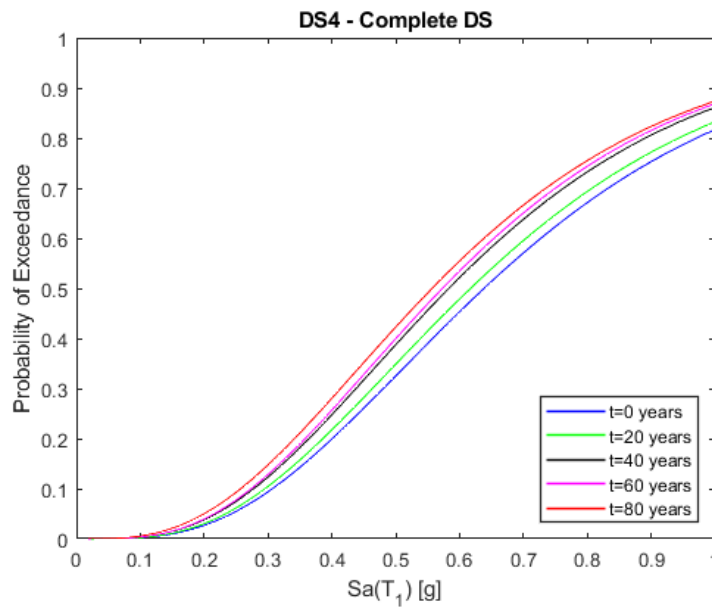


Figure 6.3. Time-variant Complete DS Fragility Curves.

Table 6.4. Median values of Fragility Curves for each DS. The values reported into the branches represent the percentual reduction with respect to the initial condition.

| DS  | Initial        | 20 years             | 40 years             | 60 years              | 80 years              |
|-----|----------------|----------------------|----------------------|-----------------------|-----------------------|
| DS1 | 0.159 g<br>(-) | 0.159 g<br>(0.00 %)  | 0.159 g<br>(0.00 %)  | 0.159 g<br>(0.00 %)   | 0.159 g<br>(0.00 %)   |
| DS2 | 0.441 g<br>(-) | 0.431 g<br>(-2.31 %) | 0.421 g<br>(-4.58 %) | 0.402 g<br>(-8.93 %)  | 0.392 g<br>(-11.04 %) |
| DS3 | 0.571 g<br>(-) | 0.564 g<br>(-1.17 %) | 0.532 g<br>(-6.80 %) | 0.508 g<br>(-11.05 %) | 0.484 g<br>(-15.14 %) |
| DS4 | 0.642 g<br>(-) | 0.620 g<br>(-3.46 %) | 0.584 g<br>(-8.96 %) | 0.571 g<br>(-11.06 %) | 0.558 g<br>(-13.12 %) |



### 6.3.2 Time-Variant Seismic Resilience

The present section highlights the main outputs of the Seismic Resilience framework. Time-variant Functionality Recovery profiles have been computed for selected IM levels. A time-variant Seismic Resilience Surface has been computed to represent the variation of the resilience profile over time and has been complemented by a Seismic Resilience Map. Seismic Resilience at different IM levels has been computed and compared to directly measure the reduction of the index over time. An Equivalent Downtime metric is computed and used to provide in-depth discussion of the results. The influence of initial functionality deterioration on the results has been quantified through comparison with a version of the framework not including this effect. Finally, the probabilistic characterization of the recovery process allowed to quantify the uncertainty in the Seismic Resilience value.

#### *Time-Variant Recovery Profiles*

The first relevant output obtained through the Seismic Resilience framework previously introduced is represented by the functionality recovery profiles, used as a direct indicator of the effects of corrosion on earthquake-induced functionality loss and subsequent functionality recovery. Figure 6.4 displays the functionality recovery profiles computed for  $t=0, 20, 40, 60$  and  $80$  years for six selected IM levels. The first, and more evident, effect of corrosion on the time-variant functionality of the bridge is that the post-event functionality following an earthquake tends to decrease due to the increased vulnerability of the bridge. The direct consequence of this phenomenon is that bridge functionality is generally lower during the recovery time prior to full functionality recovery. It is also worth noting that, due to the effects of structural deterioration, a shift of  $DS_p$  can be observed. This has been observed for the IM level corresponding to  $T_R=1000$  years.

Note that the IM value associated with a certain  $T_R$  depends on the site-specific seismicity, so discussion in these terms has to be intended as specific to a given structure in a certain site and does not allow for comparison between bridges located in different sites. Despite the reduction of post-event functionality between initial time and  $t=20$  years for that specific IM level is not dramatic, the different  $DS_p$  is also associated with different recovery time and recovery functions, ultimately affecting the  $R$  value for that specific IM level in a significant way. In a more physical sense, corrosion affects the Seismic Resilience of the bridge by reducing the residual functionality immediately after an earthquake, the functionality over the recovery time and possibly increasing the duration of the recovery phase. A final consideration is the one related to the IM levels for which the concept of Seismic Resilience is more important. In fact, even if Seismic Resilience measures the capabilities of structures to recover full functionality after earthquakes, it is clear that functionality loss for extremely rare events (such as those associated with  $T_R>10000$  years) is naturally higher than for more frequent events, consequently leading to low values of  $R$ .

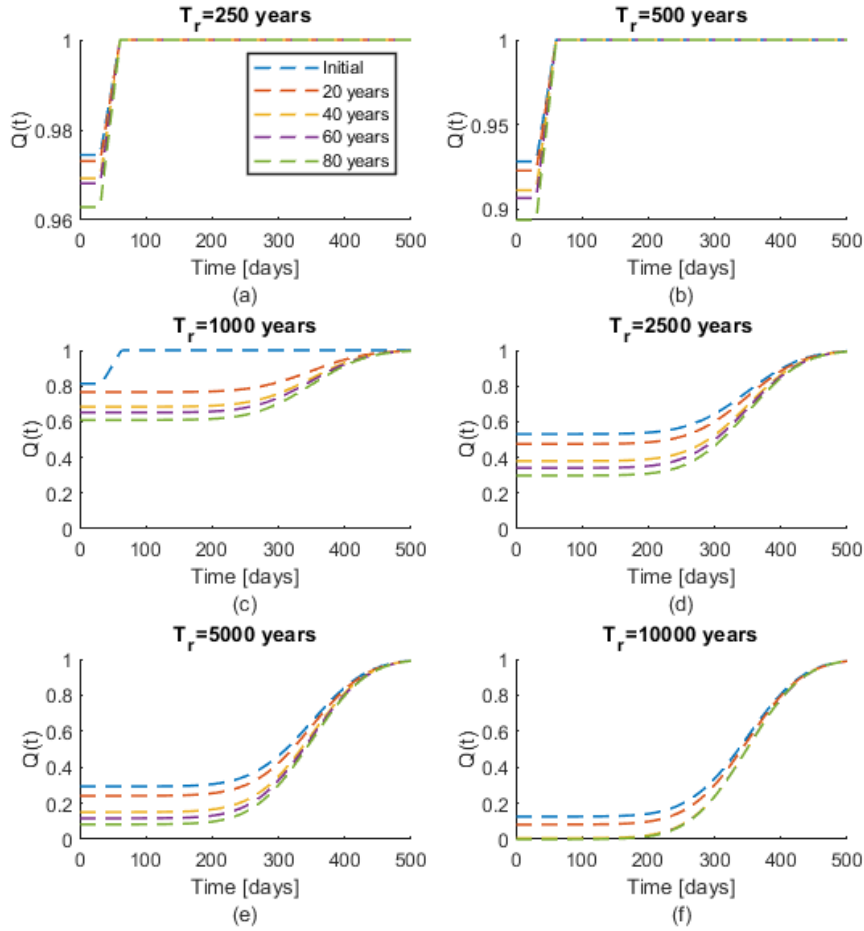


Figure 6.4. Time-variant functionality recovery paths for selected IM levels.

#### *Time-Variant Seismic Resilience Surface and Seismic Resilience Map*

First, the time-variant Seismic Resilience Surface is presented in order to show in a compact manner the deterioration of structural performance over time (Figure 6.5 (a)). The mean Seismic Resilience obtained from the Monte Carlo simulation of the recovery phase is presented. It is clear that deterioration leads to a shift of  $DS_p$ , leading to a different recovery function and, consequently, to a lower value of  $R$ . Also, the results confirm the intuitive consideration that differences in Seismic Resilience are more evident as time passes, due to more pronounced deterioration of the structural elements. It has to be pointed out, however, that initial functionality reduction plays a relevant role in the process, and that considering maintenance or retrofit interventions may lead to significantly different results.

Also, it has been assumed that  $Q_r$  is equal to 1, which translates into recovering the initial functionality of the pristine structure. This may not be always true for deteriorating structures, because economic considerations may lead to considering this  $Q_r$  value inconvenient. In this sense, the Seismic Resilience assessment carried out in the present investigation may represent an upper bound estimation. The results have been also presented in terms of Seismic Resilience Map (Figure 6.5 (b)), allowing to efficiently visualize the progressive reduction of  $R$  values over time for a given IM value. The effects of structural deterioration are more impactful for IM levels between 0.35 and 1 g, where the map shows more pronounced reduction of  $R$  values. This result can be explained by considering that IM levels below 0.35 g are characterized by low levels of seismic losses even including corrosion effects while, conversely, IM levels over 1 g are characterized by significant seismic losses even with the bridge in pristine conditions.

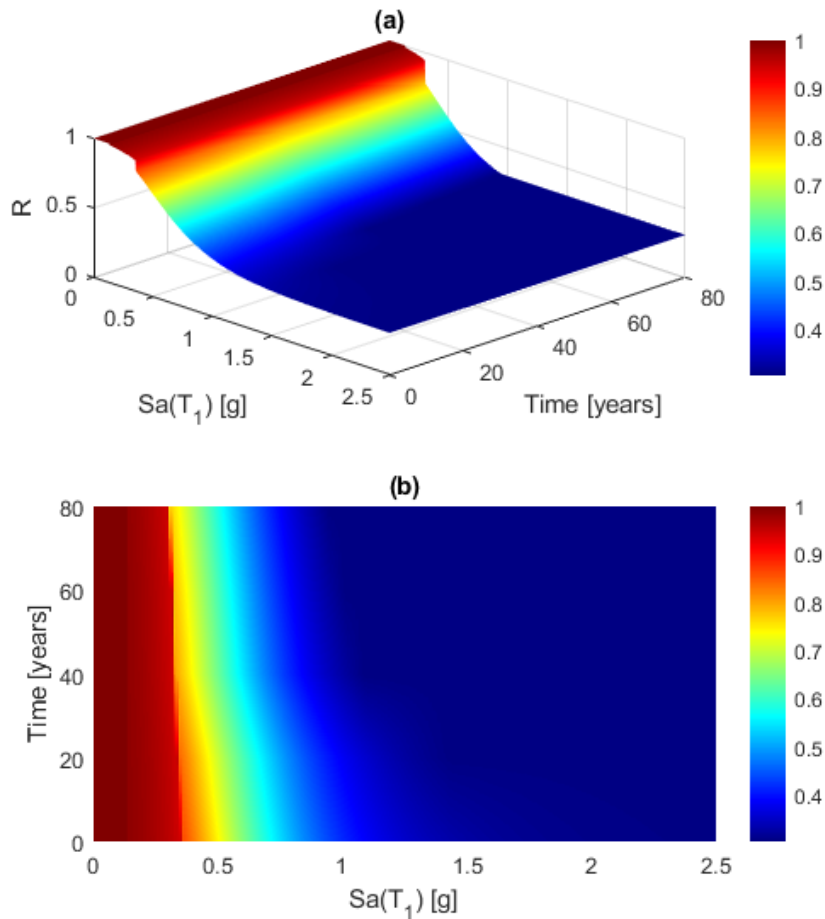


Figure 6.5. (a) Time-variant Seismic Resilience Surface and (b) time-variant Seismic Resilience Map.

### *Time-Variant Seismic Resilience for Selected IM Values*

Time-variant mean  $R$  values have been computed and compared for the 10 IM levels considered in the MSA. Figure 6.6 and Figure 6.7 display the  $R$  values over time for the 10 IM levels, expressing for clarity IM as  $S_a(T_1)$  and corresponding  $T_R$ . The results provide immediate confirmation of the relative importance of deterioration phenomena over time for different IM levels. Up to 0.23 g (corresponding to a  $T_R$  of 500 years), deterioration effects on the estimation of  $R$  are almost negligible. The relevant decrease of  $R$  observed for  $T_R=1000$  years between initial time and  $t=20$  years confirms what has been previously observed discussing the functionality recovery paths. For IM levels up to 1.08 g ( $T_R=10000$  years), a progressive reduction of  $R$  can be observed. This investigation on the values of  $R$  for selected IM levels confirms two intuitive properties of the Seismic Resilience of deterioration bridge structures: on one hand, seismic resilience decreases as IM level increases and on the other hand structural deterioration leads to decreasing seismic resilience over time for a given IM level.

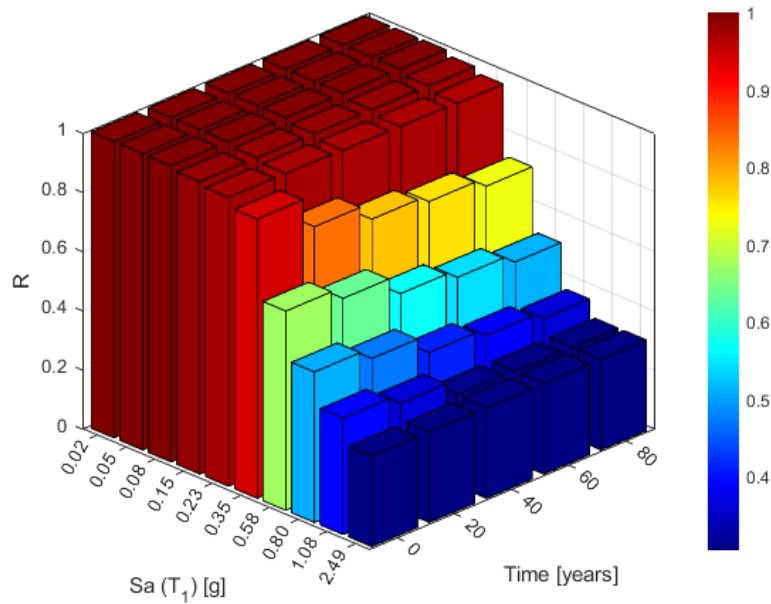


Figure 6.6. Time variant seismic resilience for selected IM values, expressed in terms of  $S_a(T_1)$ .

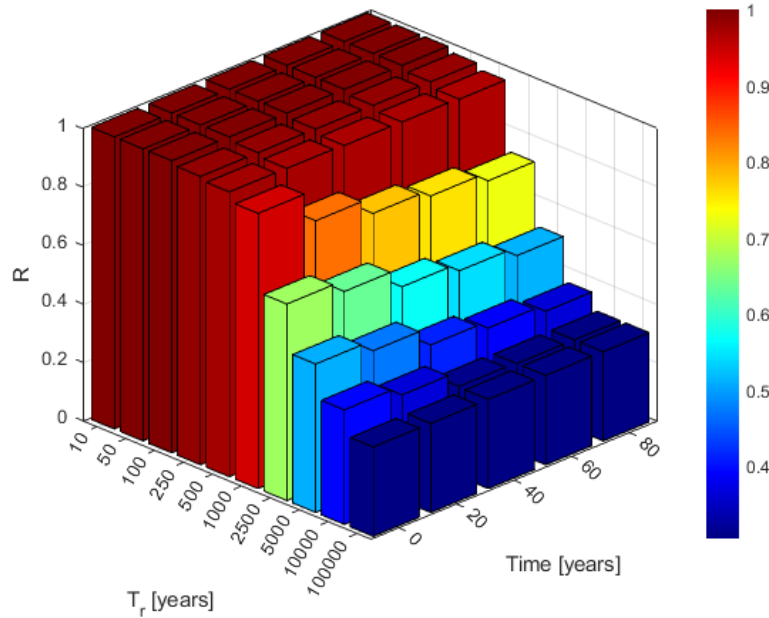


Figure 6.7. Time variant seismic resilience for selected IM values, expressed in terms of  $T_R$ .

#### Equivalent Downtime

Further characterization of time-dependent Seismic Resilience is given by estimating the Equivalent Downtime  $T_d$  for a given IM level, according to Equation (6.25) [48]:

$$(6.25) \quad T_d = \int_{t_0}^{t_h} [1 - Q(t)|IM] dt$$

Note that  $T_d$  physically represents the full closure of the bridge for a certain number of days. Table 6.5 reports mean  $R$  and  $T_d$  values computed for seven selected IM levels. The results allow us to gain some interesting insights on the effects of deterioration on the Seismic Resilience of a structure. It can be observed that the progressive reduction of  $R$  values over time is associated with increasing  $T_d$  values. The increase of  $T_d$  is dramatic for  $T_R = 1000$  years, between initial time and  $t = 20$  years, consistently with the previous findings. Presenting the results also in terms of  $T_d$  represents a direct way to measure the Seismic Resilience of a structure, providing stakeholders and analysts with a physical and easily understandable representation of the quantity for a given IM level, complementing the more analytical and synthetic nature of  $R$ .

Table 6.5.  $R$  and  $T_d$  values over time for selected IM values.

| $S_a(T_1)$<br>[g]<br>( $T_R$ )<br>[years] | Initial |                 | 20 years |                 | 40 years |                 | 60 years |                 | 80 years |                 |
|-------------------------------------------|---------|-----------------|----------|-----------------|----------|-----------------|----------|-----------------|----------|-----------------|
|                                           | R       | $T_d$<br>[days] | R        | $T_d$<br>[days] | R        | $T_d$<br>[days] | R        | $T_d$<br>[days] | R        | $T_d$<br>[days] |
| 0.15<br>(250)                             | 0.99    | 1               | 0.99     | 1               | 0.99     | 1               | 0.99     | 1               | 0.98     | 2               |
| 0.23<br>(500)                             | 0.98    | 3               | 0.97     | 3               | 0.97     | 4               | 0.97     | 4               | 0.97     | 5               |
| 0.35<br>(1000)                            | 0.94    | 9               | 0.84     | 81              | 0.78     | 110             | 0.76     | 121             | 0.73     | 135             |
| 0.58<br>(2500)                            | 0.67    | 162             | 0.63     | 182             | 0.57     | 215             | 0.54     | 228             | 0.51     | 243             |
| 0.80<br>(5000)                            | 0.51    | 245             | 0.47     | 263             | 0.41     | 294             | 0.39     | 306             | 0.36     | 318             |
| 1.08<br>(10000)                           | 0.39    | 302             | 0.36     | 318             | 0.31     | 345             | 0.31     | 346             | 0.31     | 346             |
| 2.49<br>(100000)                          | 0.31    | 346             | 0.31     | 346             | 0.31     | 346             | 0.31     | 346             | 0.31     | 346             |

#### 6.4 WHAT HAPPENS IF WE DO NOT INCLUDE $Q_0$ DETERIORATION?

One of the most important facets of time-dependent resilience assessment is that considering deterioration of structural elements should lead to the obvious consideration of reduced initial functionality for the analyzed structures. The proposed framework addresses the problem in a systematic way, based on the fragility information obtained from the MSA results.

In order to quantify the effective impact of  $Q_0$  deterioration, the mean Seismic Resilience Curves obtained considering and not considering it have been computed and compared for the different time instants considered in the study, as displayed in Figure 6.8. To quantitatively compare the difference between the curves, an  $\alpha$  value corresponding to the ratio between  $R$  values obtained not considering and considering  $Q_0$  deterioration has been computed for each IM value, and the corresponding curve has been defined. As can be expected, differences are more important as time passes and become more relevant as the probable DS shifts to DS4. The maximum  $\alpha$  values are equal to 1.08, 1.20, 1.25 and 1.30 for 20, 40, 60 and 80 years respectively. It is interesting to point out that discrepancies are more important in the central part of the Seismic Resilience curve, where the level of seismic loss progressively increases. At the same time, the results are generally aligned in

the extreme tails of the curve. This result can be explained, based on Figure 6.4, by the fact that on the left tail the level of seismic losses is low and recovery time is short, while on the right tail the level of losses leads to almost complete loss of functionality, governing the recovery process. In the central part of the curve, the level of seismic losses progressively increases but complete loss of functionality is not reached and, for this reason, the effect of  $Q_0$  deterioration is evident because it leads to lower residual functionality with respect to the case neglecting it. The results obtained in this paragraph confirm that not considering  $Q_0$  deterioration may be unconservative and not represent correctly the seismic resilience of a deteriorating structure.

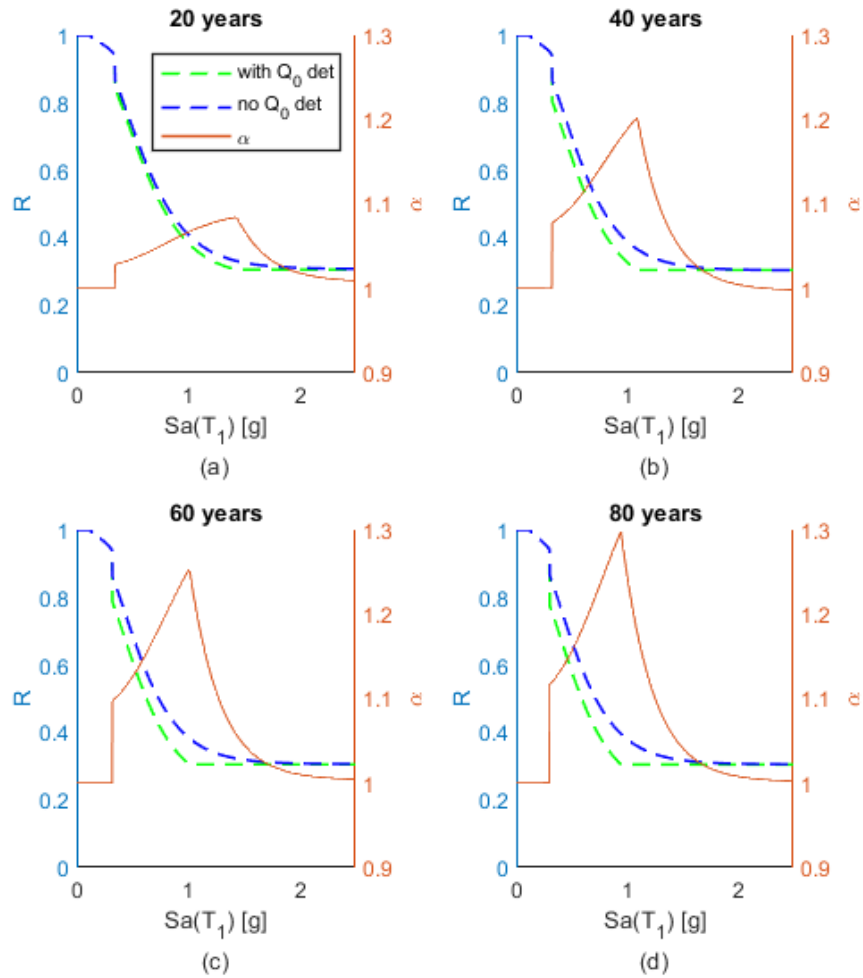


Figure 6.8. (a-d) Seismic Resilience Curves including and not including  $Q_0$  deterioration for  $t=20, 40, 60$  and  $80$  years and corresponding  $\alpha$  vs IM curves.

## 6.5 UNCERTAINTY IN SEISMIC RESILIENCE QUANTIFICATION

Previous research [12] investigated the uncertainty in Seismic Resilience estimation, finding that Seismic Resilience can be characterized through a normal distribution. Significant uncertainty has been found to characterize the estimation process, due to aleatory variation of recovery and control times.

The uncertainty on the Seismic Resilience Curves due to the aleatory components of the recovery phase is quantified and mean, 16<sup>th</sup> and 84<sup>th</sup> percentile Seismic Resilience Curves are displayed in Figure 6.9 (a)-(e). The maximum uncertainty, identified on the right tail of the curve, is  $R \pm 0.1$ . Uncertainty is naturally higher on the tail of the curve where  $DS_p$  corresponds to Complete DS, due to the longer recovery time with respect to other DS and to the higher level of functionality loss due to a seismic event. It is worth noting that corrosion leads to a progressive shift of the Seismic Resilience Curves towards lower IM levels, corresponding to a reduction of the Seismic Resilience of the bridge, as already discussed in the previous paragraphs. Further investigation has been carried out by computing the Coefficient of Variation (CV) corresponding to each value of IM, for each of the times considered in the assessment (Figure 6.9 (f)). The results reveal that Seismic Resilience reduction due to structural deterioration is associated with progressive increase of uncertainty for a given IM level, due to the increase of seismic losses correlated with more pronounced seismic vulnerability. The maximum value of CV observed in the estimation of  $R$  is around 0.36. A different perspective on the issue may be related to the reliability of the estimation. It is clear that, assuming to accept only results up to a fixed level of uncertainty, measured through CV, the range of IM levels over which Seismic Resilience estimation can be used for decision-making reduces over time.

Finally, insights on the statistical characterization of  $R$  can be obtained by observing the realizations of the value obtained through the Monte Carlo simulation, reported in Figure 6.10 for three selected IM levels. The results indicate that  $R$  values are normally distributed, even when corrosion effects are included. Additionally, corrosion effects can be observed by considering the progressive shift of the distributions towards lower values. The increasing uncertainty in the estimation due to corrosion is represented by the progressive widening of the distributions. The results obtained in the present investigation are therefore well aligned with those obtained in [12], both in terms of the type of distribution for  $R$  and maximum level of expected uncertainty.



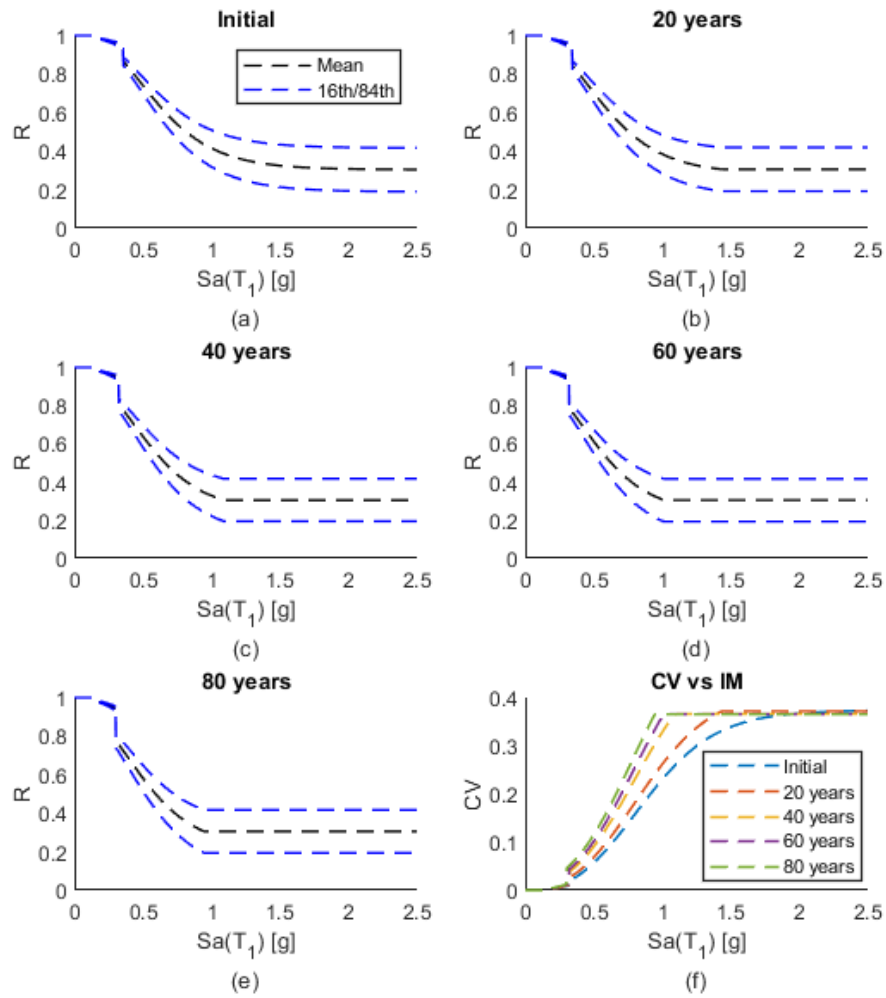


Figure 6.9. (a-e) Mean, 16<sup>th</sup> and 84<sup>th</sup> percentile Seismic Resilience Curves for initial time, 20, 40, 60 and 80 years, (f) CV vs IM.

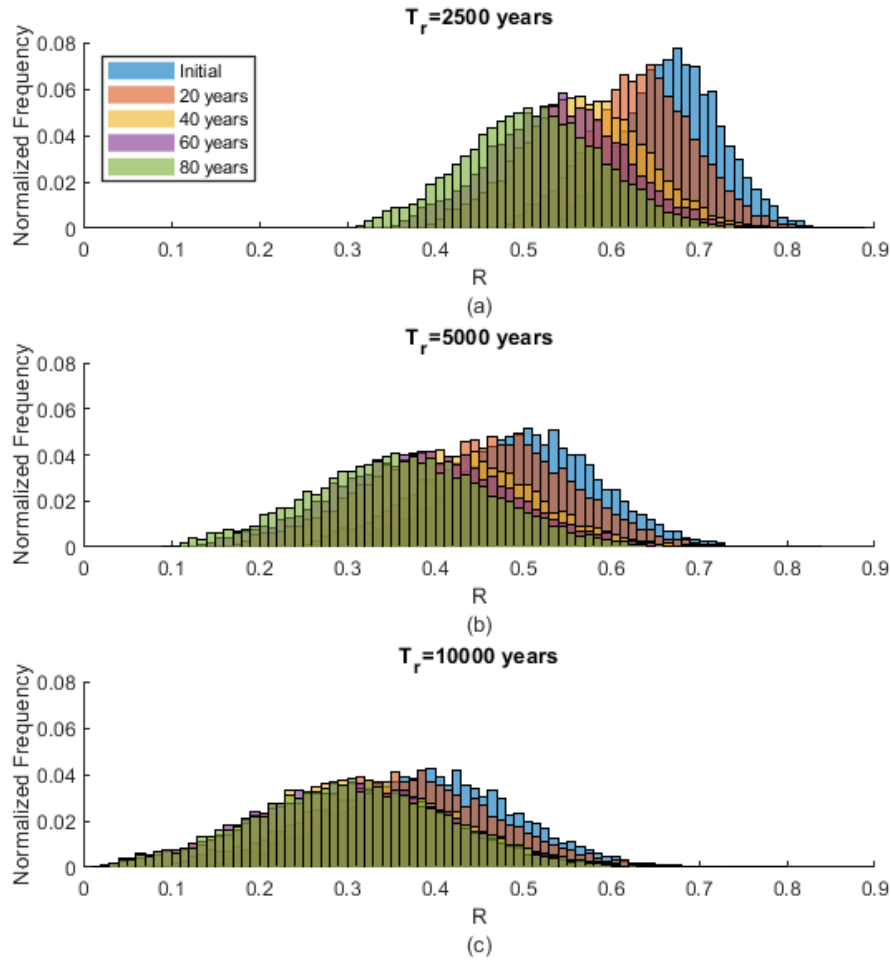


Figure 6.10. Normalized frequency of  $R$  values over time for IM levels corresponding to  $T_R$  equal to (a) 2500 years, (b) 5000 years and (c) 10000 years.

## 6.6 CONCLUSIONS AND FUTURE DEVELOPMENTS

This study presented a Seismic Resilience framework tailored for deteriorating bridge structures. The components of the framework have been introduced. Amongst the main features of the framework, time variant  $Q_0$  and probabilistic characterization of the recovery process have been defined. An example application of the framework is the provided. The corrosion process of a case study bridge has been studied through Monte Carlo simulation. Fragility analysis has been carried out according to the state-of-the-art PBEE approaches. Finally, Seismic Resilience curves have been computed for five different time steps. The results highlight that Seismic Resilience decreases significantly over time, as shown by

Seismic Resilience Surface, Seismic Resilience Map and by comparison of  $R$  values for selected IM levels. Further characterization of the evolution of Seismic Resilience over time is given by estimating the Equivalent Downtime  $T_d$ . Presenting the results also in terms of  $T_d$  represents a direct way to measure the Seismic Resilience of a structure, providing stakeholders and analysts a physical and easily understandable representation of the quantity. Including the initial functionality deterioration is necessary in order to describe realistically the time-dependent seismic performance of a bridge, as found comparing Resilience Curves computed with and without including  $Q_0$  deterioration. Finally, the uncertainty in Seismic Resilience assessment has been quantified, highlighting a maximum uncertainty of  $R \pm 0.1$ , aligned with literature results. Uncertainty is higher on the right portion of the curve where the probable DS is DS4, characterized by the highest recovery time, and the expected seismic losses are higher due to the probability of reaching DS4.

Future developments of the framework, designed with modular philosophy and readily updatable over time, may include incorporating the effects of retrofit and maintenance, providing robust and theoretically sound support to the management of bridge structures. Investigations related to different target functionality for the recovery process are necessary when dealing with deteriorating bridge structures. Finally, as more experimental evidence becomes available, corrosion damage modeling may be extended, further improving the mechanical representation of deterioration effects.

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## **7. SCENARIO-BASED LONG-TERM SEISMIC RESILIENCE OF EXISTING BRIDGE CONFIGURATIONS**

### **7.1 INTRODUCTION**

As widely known by the engineering community, bridge structures represent crucial nodes of infrastructure networks all over the world. Most of the existing bridge structures were built around 60 years ago, and for this reason may present severe deterioration phenomena. Specifying the discussion to the Italian context, most of the existing bridges have been built using Reinforced Concrete (RC) or Prestressed Concrete (PC) [1]. At the time, durability and seismic design issues were addressed in approximate way, compared to modern approaches, and for this reason maintenance and retrofit represent crucial facets of bridge management. The rise in bridge collapses frequency identified in the literature [2] indicates the relevance of deterioration phenomena and lack of maintenance. Given the contemporary shortage of resources for bridge maintenance and increase of traffic volumes [3], bridge assessment should be guided by rational and theoretically consistent approaches in order to optimize the management process. In this sense, Seismic Resilience represents an appealing concept for the assessment of bridges and networks [4], being a transparent and holistic approach and facilitating communication between engineers and stakeholders.

This chapter addresses the long-term Seismic Resilience assessment of existing bridge structure configurations. The framework introduced in Chapter 6 is applied to four existing bridge configurations inspired by a real case study bridge, in order to study both regular and irregular structural schemes. Scenario-based analysis is carried out assuming, for each configuration, different chloride exposure conditions. The goals of the chapter are therefore: i) investigating the influence of exposure conditions on the long-term Seismic Resilience assessment of bridge structures; ii) studying different structural configurations representative of existing Italian bridges in order to provide insights on the inter-configuration variability of Seismic Resilience metrics.

### **7.2 METHODOLOGY**

The present section briefly recalls, for the sake of convenience, the main components of the Seismic Resilience assessment framework presented in Chapter 6. The framework is specified to existing Italian bridge configurations and, for this reason, some slight differences with the general framework are introduced and discussed.

### 7.2.1 Seismic Fragility Analysis

Multiple Stripe Analysis (MSA) is performed to characterize the Engineering Demand Parameter (EDP) distribution conditional on Intensity Measure (IM) [5]. 10 IM levels with return period  $T_R$  10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 are considered for the analysis. The spectral acceleration at the first mode period  $S_a(T_1)$  is used as IM [6][7]. The site-specific seismic hazard curve is computed through REASSESS [8]. Hazard consistent ground motions are selected for each IM level through the Conditional Spectrum approach [9][10], matching mean and variance of the distribution. 20 pairs of ground motions are retrieved from the PEER database [11] for each IM level.

Four Damage States (DS), namely Slight (DS1), Moderate (DS2), Extensive (DS3) and Complete (DS4) are identified for seismic fragility analysis. The thresholds considered for identifying the exceedance of each DS for the bridge components have been retrieved from [12] and are reported in Table 7.1. Note that unbolted neoprene pad bearings represent the most common bearing devices in existing Italian bridges.

**Table 7.1. Thresholds for each component and for each DS.  $\Delta_y$ : yielding displacement;  $\Delta_u$ : ultimate displacement;  $d_{fr}$ : displacement corresponding to the attainment of friction resistance;  $d_{pad}$ : pad dimension in motion direction;  $d_{uns}$ : displacement corresponding to deck unseating;  $d_{gap}$ : gap between deck and abutment backwall;  $d_{y,ab}$ : displacement corresponding to development of system lateral strength;  $d_{u,ab}$ : ultimate backwall displacement.**

| Component                      | EDP          | DS1        | DS2                                   | DS3                                             | DS4        |
|--------------------------------|--------------|------------|---------------------------------------|-------------------------------------------------|------------|
| Piers                          | Displacement | $\Delta_y$ | $\Delta_y + 0.5(\Delta_u - \Delta_y)$ | $\Delta_y + 2/3(\Delta_u - \Delta_y)$           | $\Delta_u$ |
| Unbolted Neoprene Pad Bearings | Displacement | $d_{fr}$   | $d_{fr} + 1/3(d_{pad} - d_{fr})$      | $d_{pad}$                                       | $d_{uns}$  |
| Backwall                       | Displacement | $d_{gap}$  | $d_{y,ab}$                            | $\Delta_y + 2/3(\Delta_{u,ab} - \Delta_{y,ab})$ | $d_{u,ab}$ |

Fragility curves are defined for each DS applying the Maximum Likelihood Estimation (MLE) method [13]. The bridge is assumed to behave as a series system, using as EDP the maximum displacement demand/capacity ratio between those of the different bridge components. The total dispersion  $\beta_{comp}$  is computed considering record-to-record variability (estimated through the MLE method) and modeling uncertainty (computed as per [14], assuming average construction and model quality).

### 7.2.2 Seismic Resilience Assessment

Direct Seismic Losses are estimated according to the HAZUS approach [15]. Table 7.2 reports the Damage Ratios  $\mu_{D,i}$  for each DS.

Table 7.2. Damage Ratios for each DS according to [15].

| Damage State | Damage Ratio                                              |
|--------------|-----------------------------------------------------------|
| DS1          | 0.03                                                      |
| DS2          | 0.08                                                      |
| DS3          | 0.25                                                      |
| DS4          | 1 or 2/number of spans (for bridges with 3 or more spans) |

The approach proposed in [16] is implemented to identify the probable Damage State of the bridge  $DS_p$  for a given IM level and the consequent recovery phase. The pre-event functionality at a given time  $Q_0(t)$  is computed, after identifying  $DS_p$ , based on Equation (7.2):

$$(7.2) \quad Q_0(t_i) = \frac{\theta_{DS_p}(t_i)}{\theta_{DS_p}(t_0)}$$

where  $\theta_{DS_p}(t_i)$  is the median of the fragility curve corresponding to  $DS_p$  at the time  $t_i$ , while  $\theta_{DS_p}(t_0)$  is the median of the same fragility curve for the bridge in pristine condition.

The general expression of the post-event functionality recovery model  $Q(\tau)$  is described through Equation (7.3) [17]:

$$(7.3) \quad Q(\tau) = Q_r + H(\tau)f_{rec}(\tau)(Q_t - Q_r)$$

Where  $Q_r$  is the post-event residual functionality defined as  $Q_0(t)-L$ ,  $H(\cdot)$  is the Heaviside step function,  $\tau$  is a normalized time defined as  $(t-t_i)/(t_{rec}-t_i)$  and  $f_{rec}$  is the recovery function.

Restoration models for each DS (as described in Chapter 6) have been retrieved from the literature [17]-[19] and have been reported in Equations (7.4)-(7.6):

$$(7.4) \quad f_{rec}(\tau) = 1 - e^{-\alpha\tau} \text{ negative - exponential}$$

$$(7.5) \quad f_{rec}(\tau) = \frac{1-\cos(\pi\tau)}{2} \text{ sinusoidal}$$

$$(7.6) \quad f_{rec}(\tau) = e^{-\alpha(1-\tau)} \text{ positive - exponential}$$

where  $\alpha$  is a shape parameter of the recovery functions.  $t_{rec}$  is defined based on the HAZUS restoration curves for highway bridges [15][20], assumed to be applicable to the Italian environment.  $\alpha$ ,  $\Delta_i$  and  $t_{rec}$  are characterized as Random Variables (RVs) with distributions specified in Table 7.3.

**Table 7.3.** Statistical Distributions of recovery parameters.  $\mu$ =mean;  $\sigma$ =standard deviation.

| Parameter              | Distribution | Slight DS                    | Moderate DS              | Extensive DS                 | Complete DS             |
|------------------------|--------------|------------------------------|--------------------------|------------------------------|-------------------------|
| $\alpha$ [18]          | Uniform      | [8-10]                       | -                        | [6-8]                        | [8-10]                  |
| $t_{rec}$ [20] [days]  | Normal       | $\mu=1.2$ ,<br>$\sigma=0.35$ | $\mu=5.2$ , $\sigma=1.9$ | $\mu=117$ ,<br>$\sigma=24.2$ | $\mu=340$ , $\sigma=63$ |
| $\Delta_i$ [21] [days] | Uniform      | [30-60]                      | [30-60]                  | [30-60]                      | [30-60]                 |

The Seismic Resilience Index  $R$  is computed according to Equation (7.7):

$$(7.7) \quad R = \frac{1}{t_h - t_0} \int_{t_0}^{t_h} Q(t) dt$$

Where  $t_h$  is the control time,  $t_0$  is the earthquake occurrence time and  $Q(t)$  is the bridge functionality.  $t_h$  is taken as 170 days for Slight, Moderate and Extensive DS, and it is taken as 500 days for Complete DS [15].

Equivalent Downtime  $T_d$  for a given IM level is computed according to Equation (7.8) [22]:

$$(7.8) \quad T_d = \int_{t_0}^{t_h} [1 - Q(t)|IM] dt$$

$T_d$  is computed to provide further characterization of Seismic Resilience over time and physically represents the full closure of the bridge for a certain number of days.

### 7.2.3 Corrosion Model

The error function solution of 1D Fick's Second Law of Diffusion is used to describe the time-dependent chloride concentration [23][24]. Corrosion starts once the critical concentration  $C_{crit}$  of free chloride ions is reached at depth  $x$ . Chloride diffusion process is simulated through Monte Carlo simulation. Corrosion propagation is characterized through Faraday's Law [25]. Localized corrosion is described through the pitting corrosion model described in [26].  $R$  is modeled through a Gumbel distribution [27] and the time-dependent value of  $i_{corr}(t)$  is estimated as per [28].

Steel material properties (yield strength  $f_y$ , ultimate strength  $f_u$  and ultimate strain  $\varepsilon_{su}$ ) have been computed according to the deterioration models proposed in [29][30]. Cover concrete compressive strength is reduced through the model proposed in [31], based on data reported in [32] and [33]. Confined concrete compressive strength  $f_c^c$  and ultimate strain  $\varepsilon_{cu}$  for corroded elements can be computed as per [34].

### 7.3 STRUCTURAL CONFIGURATIONS

Four existing bridge configurations derived from a real case study have been considered in the present investigation. Configuration 1 (C1) represents the original bridge configuration and corresponds to Bridge 3 in Table 1.1 and Table 1.2. Configuration 2 (C2, corresponding to Bridge 4 in Table 1.1 and Table 1.2) includes pier height irregularity but is characterized by straight deck configuration. Conversely, Configuration 3 (C3, referred to as Bridge 5 in Table 1.1 and Table 1.2) has piers with equal height, but includes deck skewness. Finally, Configuration 4 (C4) represents a completely regular configuration, with equal height piers and straight deck and is indicated as Bridge 6 in Table 1.1 and Table 1.2.

The original case study bridge is located in Parma (Italy) and has been built in 1969. The bridge includes four 30 m long spans and is characterized by a skew deck. The deck is made of four PC beams resting on 49 mm thick unbolted neoprene pad bearings, completed by a 0.25 m thick slab. The height of the piers is 7.71, 13.31 and 17.91 m respectively. Piers are composed of two 3300x700 mm RC walls connected on their upper portion by a 0.6 m thick slab. The plastic hinge zone of the walls is reinforced with 0.24%, 0.36% and 0.49% of longitudinal rebars for piers with height of 7.71, 13.31 and 17.91 m respectively. Stirrups with 8 mm diameter and 200 mm spacing have been used as transverse reinforcement. The nominal concrete cover value  $c_{nom}$  is equal to 43 mm. Assuming average construction quality, the water to cement ratio  $w/c$  has been assumed to be 0.5. The initial value of  $f_c$  is equal to 32.5 MPa. The initial value of  $f_f$  is 434 MPa, according to the properties of steel at the time [35]. A summary of the properties of the four structural configurations is reported in Table 7.4, and their schematization can be found in Figure 7.1 and Figure 7.2.

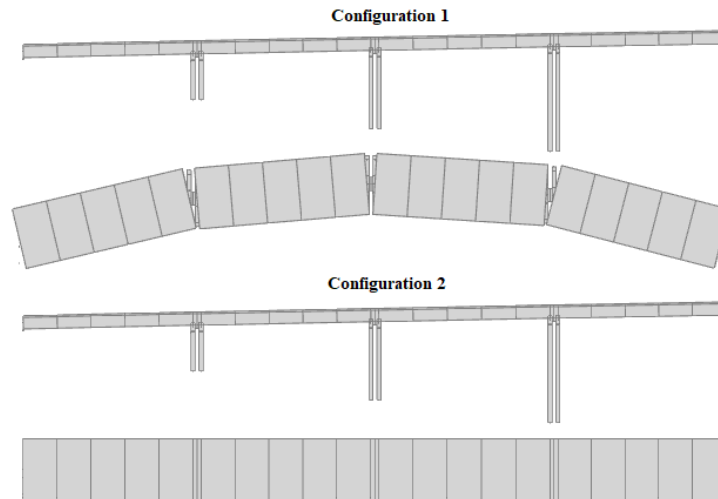


Figure 7.1. Structural schemes of Configuration 1 and Configuration 2.

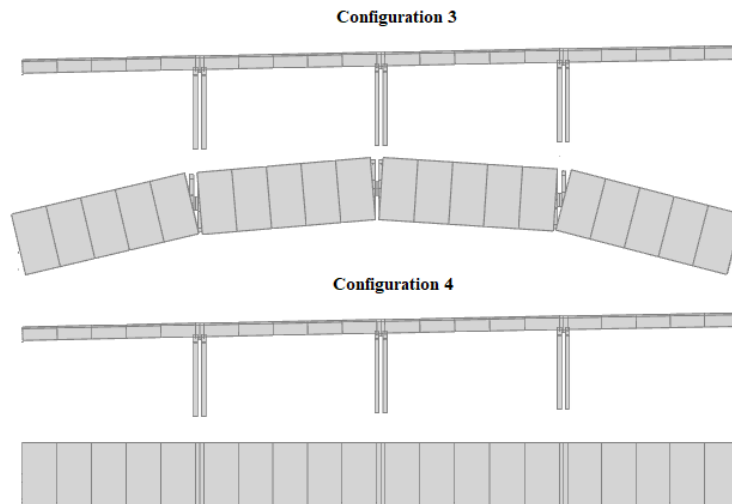


Figure 7.2. Structural schemes of Configuration 3 and Configuration 4.

Table 7.4. Summary of the properties of the four configurations.

| Configuration | Height Pier 1<br>[m] | Height Pier 2<br>[m] | Height Pier 3<br>[m] | Skew |
|---------------|----------------------|----------------------|----------------------|------|
| C1            | 7.71                 | 13.31                | 17.91                | Yes  |
| C2            | 7.71                 | 13.31                | 17.91                | No   |
| C3            | 13.31                | 13.31                | 13.31                | Yes  |
| C4            | 13.31                | 13.31                | 13.31                | No   |

### 7.3.1 Numerical Modeling

The numerical models of the four structural configurations have been developed in OpenSees [36]. The superstructure has been assumed to respond elastically to the seismic action and has been modeled using five spine elements for each span. Unbolted neoprene pad bearings have been modeled through *TwoNodeLink* elements, assuming a neoprene-concrete friction coefficient of 0.4 [37]. Bent caps have been modeled using elastic beams, while distributed inelasticity elements with force-based formulation and Gauss-Radau integration scheme have been used for the walls [38], computing the plastic hinge length as per [39]. Fiber sections have been defined using the *Steel02* uniaxial material for reinforcement fibers, coupled with a *MinMax* criterion to capture bar fractures. Cover concrete fibers have been described through the *Concrete01* material, while core fibers have been characterized using the *Concrete04* material. The model of Mander et al. [40] has been used to compute confined concrete mechanical properties. The interaction between deck and abutment backwall has been captured through the *ElasticPPGap* material according to the indications reported in [37]. Contact between adjacent spans has been modeled through

*Impact* elements, defined according to Muthukumar and DesRoches [41]. The numerical models of the four structural configurations have been defined for initial time, 20 years, 40 years and 60 years based on the mean properties retrieved from the corrosion simulation.

### 7.3.2 Exposure Conditions

In order to provide a comprehensive scenario-based assessment, three different exposure conditions have been identified. The three scenarios are characterized by progressively more aggressive environmental exposure conditions. Scenario 1 corresponds to marine environment exposure for structures located on the coast and is comparable to the XS1 Exposure Class according to the Eurocode. Scenario 2 corresponds to permanently submerged parts of marine structures, comparable with XS2 Exposure Class. Scenario 3 refers to splash and spray zones and corresponds to XS3 Exposure Class. The parameters characterizing the statistical distributions of the RVs of Equation (2.1) have been retrieved from the literature [23][24] and are reported for completeness in Table 7.5-Table 7.7.

**Table 7.5. Characteristics of the RVs considered for Scenario 1, retrieved from [23][24].**

| RV                                         | Distribution Type | Mean ( $\mu$ ) | Standard Deviation | Bounds                     |
|--------------------------------------------|-------------------|----------------|--------------------|----------------------------|
| $D_{rm,0}$ [ $10^{-12}$ m/s <sup>2</sup> ] | Normal            | 15.8           | 0.2 $\mu$          | $\geq 0$                   |
| a [-]                                      | Normal            | 0.65           | 0.12               | $0 \leq a \leq 1$          |
| $T_{real}$ [K]                             | Normal            | 285.95         | 5                  | -                          |
| b                                          | Normal            | 4800           | 700                | -                          |
| $C_{b-lx}$ [wt-%/c]                        | Lognormal         | 2              | 0.9                | $\geq 0$                   |
| $\Delta lx$ [mm]                           | Constant          | 0              | -                  | -                          |
| $C_{crit}$ [wt-%-c]                        | Beta              | 0.6            | 0.15               | $0.2 \leq C_{crit} \leq 2$ |
| $c$ [mm]                                   | Normal            | $c_{nom}$      | 6                  | $\geq \varphi t + 1$       |
| $f_c$ [MPa]                                | Lognormal         | 32.5           | 3.6                | $\geq 0$                   |
| $f_y$ [MPa]                                | Lognormal         | 434            | 26.3               | $\geq 0$                   |

Table 7.6. Characteristics of the RVs considered for Scenario 2, retrieved from [23][24].

| RV                                         | Distribution Type | Mean ( $\mu$ ) | Standard Deviation | Bounds                     |
|--------------------------------------------|-------------------|----------------|--------------------|----------------------------|
| $D_{rm,0}$ [ $10^{-12}$ m/s <sup>2</sup> ] | Normal            | 15.8           | $0.2\mu$           | $\geq 0$                   |
| a [-]                                      | Normal            | 0.3            | 0.12               | $0 \leq a \leq 1$          |
| $T_{real}$ [K]                             | Normal            | 285.95         | 5                  | -                          |
| b                                          | Normal            | 4800           | 700                | -                          |
| $C_{s,\Delta x}$ [wt-%/c]                  | Lognormal         | 4              | 1                  | $\geq 0$                   |
| $\Delta x$ [mm]                            | Constant          | 0              | -                  | -                          |
| $C_{crit}$ [wt-%-c]                        | Beta              | 0.6            | 0.15               | $0.2 \leq C_{crit} \leq 2$ |
| $c$ [mm]                                   | Normal            | $c_{nom}$      | 6                  | $\geq \varphi_t + 1$       |
| $f_c$ [MPa]                                | Lognormal         | 32.5           | 3.6                | $\geq 0$                   |
| $f_f$ [MPa]                                | Lognormal         | 434            | 26.3               | $\geq 0$                   |

Table 7.7. Characteristics of the RVs considered for marine Scenario 3, retrieved from [23][24].

| RV                                         | Distribution Type | Mean ( $\mu$ ) | Standard Deviation | Bounds                     |
|--------------------------------------------|-------------------|----------------|--------------------|----------------------------|
| $D_{rm,0}$ [ $10^{-12}$ m/s <sup>2</sup> ] | Normal            | 15.8           | $0.2\mu$           | $\geq 0$                   |
| a [-]                                      | Normal            | 0.3            | 0.12               | $0 \leq a \leq 1$          |
| $T_{real}$ [K]                             | Normal            | 285.95         | 5                  | -                          |
| b                                          | Normal            | 4800           | 700                | -                          |
| $C_{s,\Delta x}$ [wt-%/c]                  | Lognormal         | 4              | 1.8                | $\geq 0$                   |
| $\Delta x$ [mm]                            | Beta              | 10             | 5                  | $0 \leq \Delta x \leq c-1$ |
| $C_{crit}$ [wt-%-c]                        | Beta              | 0.6            | 0.15               | $0.2 \leq C_{crit} \leq 2$ |
| $c$ [mm]                                   | Normal            | $c_{nom}$      | 6                  | $\geq \varphi_t + 1$       |
| $f_c$ [MPa]                                | Lognormal         | 32.5           | 3.6                | $\geq 0$                   |
| $f_f$ [MPa]                                | Lognormal         | 434            | 26.3               | $\geq 0$                   |

#### 7.4 RESULTS AND DISCUSSION

This section presents and discusses the results obtained through the different modules of the Seismic Resilience assessment framework. DS4 fragility curves, R values for selected IM levels, Resilience Maps and  $T_d$  values for selected IM levels are considered to discuss the long-term seismic performance of the four structural configurations in the three exposure scenarios considered in this study.



#### 7.4.1 Seismic Fragility

A first comparison between different structural configurations and exposure scenarios is carried out by observing the evolution of fragility curves over time. For the sake of brevity, only DS4 fragility curves are presented in this paragraph. Scenario 2 and Scenario 3 led to significant differences in DS4 fragility curves over time, while the effects of Scenario 1 were almost negligible for the four structural configurations. Figure 7.3 and Figure 7.4 display the evolution of DS4 fragility curves for Scenario 2 and Scenario 3, for each of the four structural configurations investigated. For the sake of completeness, Table 7.8 reports the median IM values for each curve and scenario and the percentual variation of the value with respect to initial time.

A few comments can be made based on these results. Considering the initial time and based on the median IM values, the seismic vulnerability of C1 and C4 is almost the same, C2 is the least vulnerable and C3 is the most vulnerable. The evolution of fragility curves over time allows us to get insights on effects of progressively more aggressive exposure scenarios. It is in fact evident how, for a fixed time, the increase of seismic vulnerability is more pronounced passing from Scenario 1 to Scenario 3, with a maximum reduction of median IM value of 53.07%, observed at 60 years for C4.

**Table 7.8. Median IM values for DS4, for each configuration and exposure scenario, at initial time, 20 years, 40 years and 60 years. The values between brackets represent the percentual variation of the value with respect to initial time.**

| Configuration | Scenario | Initial  | 20 years      | 40 years      | 60 years      |
|---------------|----------|----------|---------------|---------------|---------------|
| C1            | 1        | 1.08 (-) | 1.08 (-)      | 1.08 (-)      | 1.08 (-)      |
|               | 2        | 1.08 (-) | 1.08 (-)      | 0.91 (-15.74) | 0.68 (-37.06) |
|               | 3        | 1.08 (-) | 1.08 (-)      | 0.66 (-38.87) | 0.59 (-44.87) |
| C2            | 1        | 1.16 (-) | 1.16 (-)      | 1.16 (-)      | 1.16 (-)      |
|               | 2        | 1.16 (-) | 1.08 (-7.10)  | 0.91 (-21.72) | 0.64 (-44.64) |
|               | 3        | 1.16 (-) | 0.96 (-17.64) | 0.66 (-43.21) | 0.59 (-48.90) |
| C3            | 1        | 1.01 (-) | 1.01 (-)      | 1.01 (-)      | 1.01 (-)      |
|               | 2        | 1.01 (-) | 0.91 (-10.19) | 0.83 (-17.97) | 0.71 (-29.80) |
|               | 3        | 1.01 (-) | 0.83 (-17.97) | 0.54 (-46.47) | 0.48 (-52.07) |
| C4            | 1        | 1.08 (-) | 1.08 (-)      | 1.08 (-)      | 1.08 (-)      |
|               | 2        | 1.08 (-) | 0.87 (-19.59) | 0.81 (-25.09) | 0.64 (-40.40) |
|               | 3        | 1.08 (-) | 0.83 (-23.04) | 0.54 (-49.92) | 0.51 (-53.07) |

It is also worth noting that each configuration evolves in a different way over time, but each one of them is characterized by pronounced deterioration effects for Scenario 2 and Scenario 3, which is a direct consequence of the interaction between aggressive exposure scenarios and deficient durability-oriented details. Finally, it is worth pointing out that the increase of seismic vulnerability of the configurations will affect significantly the Seismic Resilience assessment. From the mathematical point of view, this is relevant because increased seismic vulnerability leads to higher levels of losses and, consequently, lower functionality levels over the recovery phase. The physical consequence of increased seismic vulnerability is higher damage sustained by the structure, increasing repair cost and repair time after an earthquake, directly affecting the post-event emergency management.

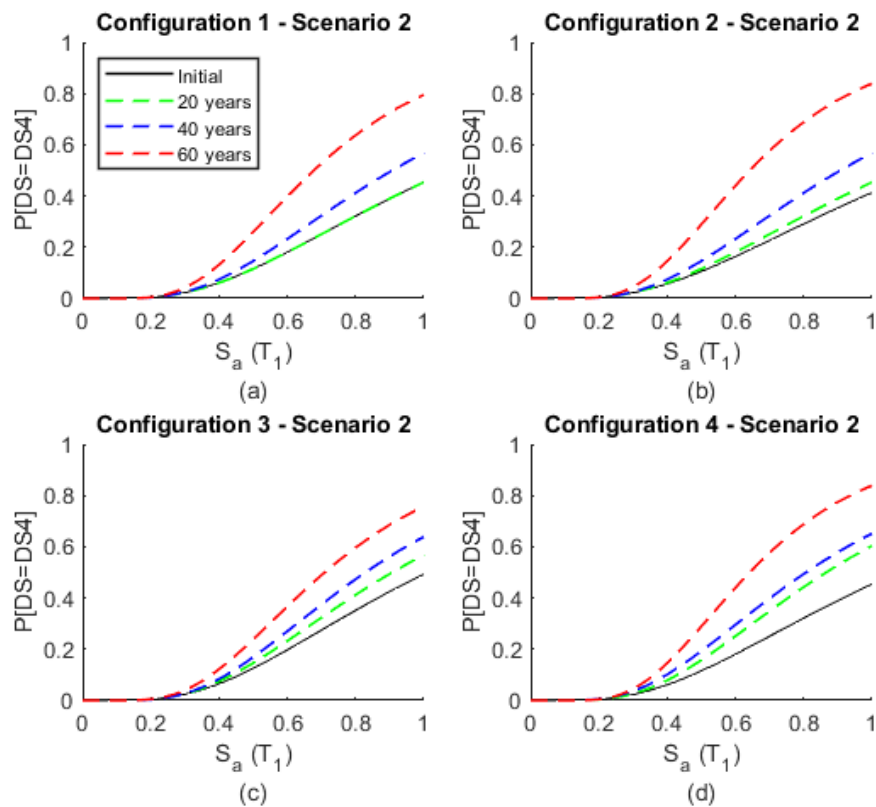


Figure 7.3. (a-d) DS4 Fragility curves at initial time, 20 years, 40 years and 60 years for C1-C4. Scenario 2.

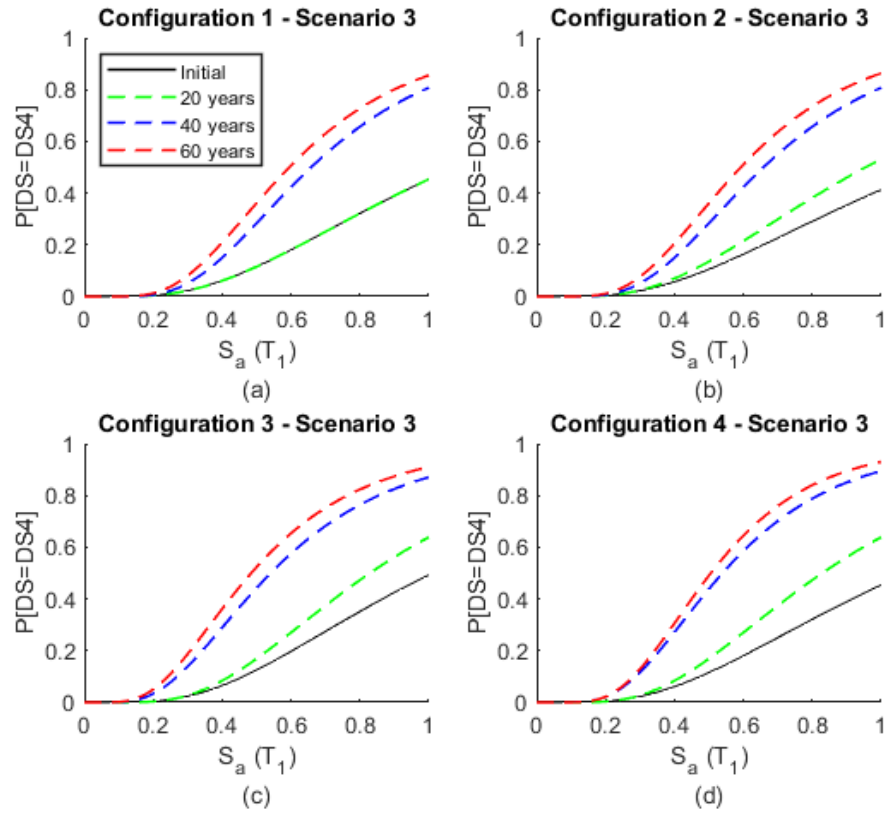


Figure 7.4. (a-d) DS4 Fragility curves at initial time, 20 years, 40 years and 60 years for C1-C4. Scenario 3.

#### 7.4.2 Seismic Resilience

Seismic Resilience assessment results are first discussed by comparing mean  $R$  values over time, for each exposure scenario and structural configuration, for IM levels 4-10. The results have been reported in Figure 7.5-Figure 7.7. The initial Seismic Resilience of each configuration is essentially the same, as can be observed by considering the  $R$  values for each IM level. The preliminary results obtained in terms of fragility analysis have been confirmed and their propagation to Seismic Resilience assessment can be observed. As expected, Seismic Resilience is inversely proportional to the IM level, which is a direct consequence of higher expected losses for higher IM levels.

The effects of progressively more severe exposure scenarios can be discussed by observing that, for the same IM level,  $R$  values are progressively lower passing from Scenario 1 to Scenario 3. It is also worth noting that deterioration effects tend to become more evident

for progressively lower IM levels as time passes, as a direct consequence of increase of seismic vulnerability and shift of  $DS_P$ , depending on the structural configuration. Deterioration effects are, however, negligible for IM levels characterized by  $T_R$  lower than 5000 years, highlighting how each structural configuration is characterized by high Seismic Resilience even for rare events (such as those characterized by  $T_R$  equal to 2500 years). Note that the IM value associated with a certain  $T_R$  depends on the site-specific seismicity, so discussion in these terms has to be intended as specific to a given structure (or set of structures, in this case) in a certain site and does not allow for comparison between bridges located in different sites.

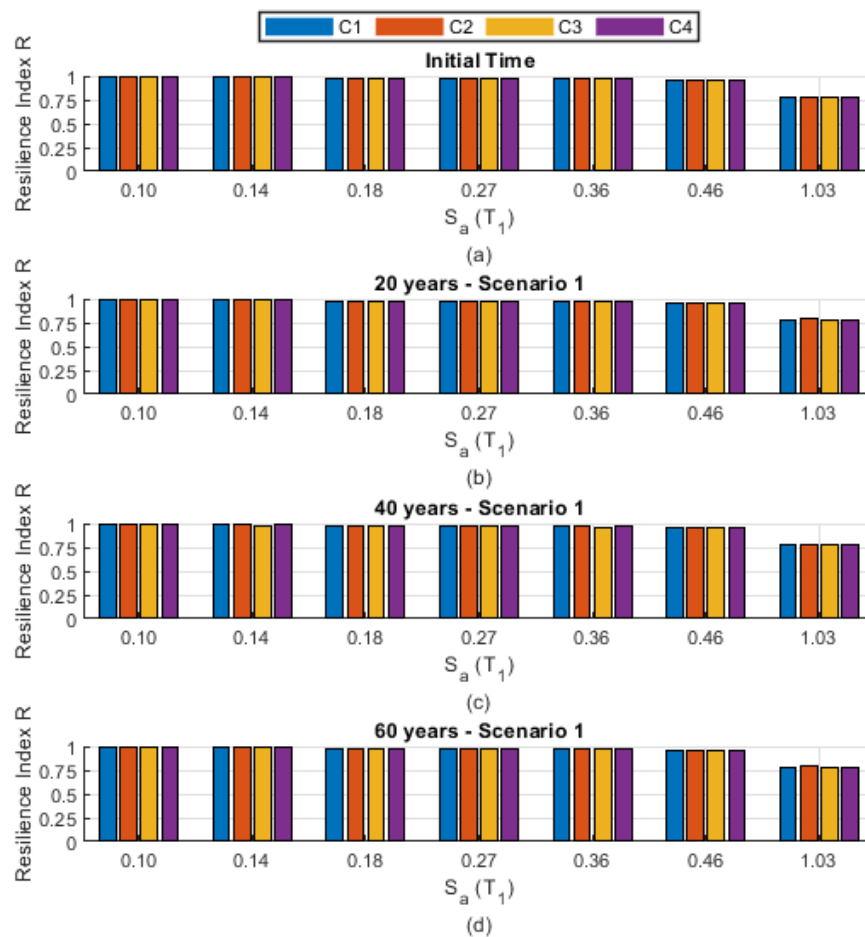


Figure 7.5.  $R$  values for selected IM levels at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years. Scenario 1.

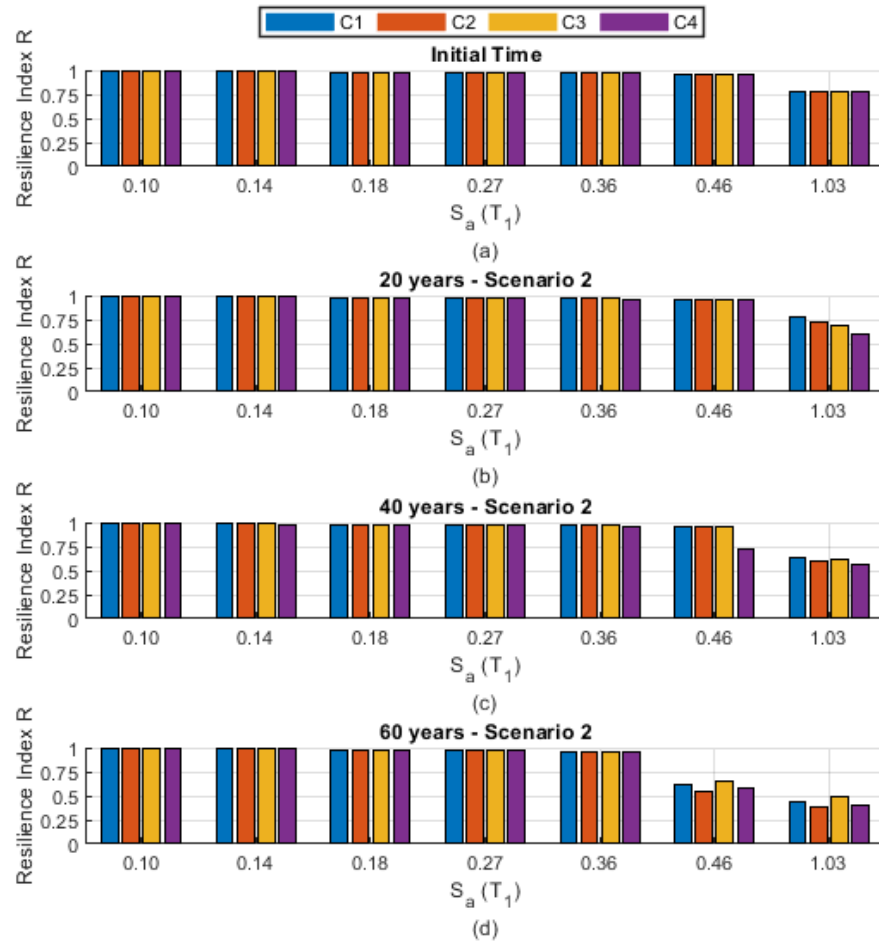


Figure 7.6.  $R$  values for selected IM levels at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years. Scenario 2.

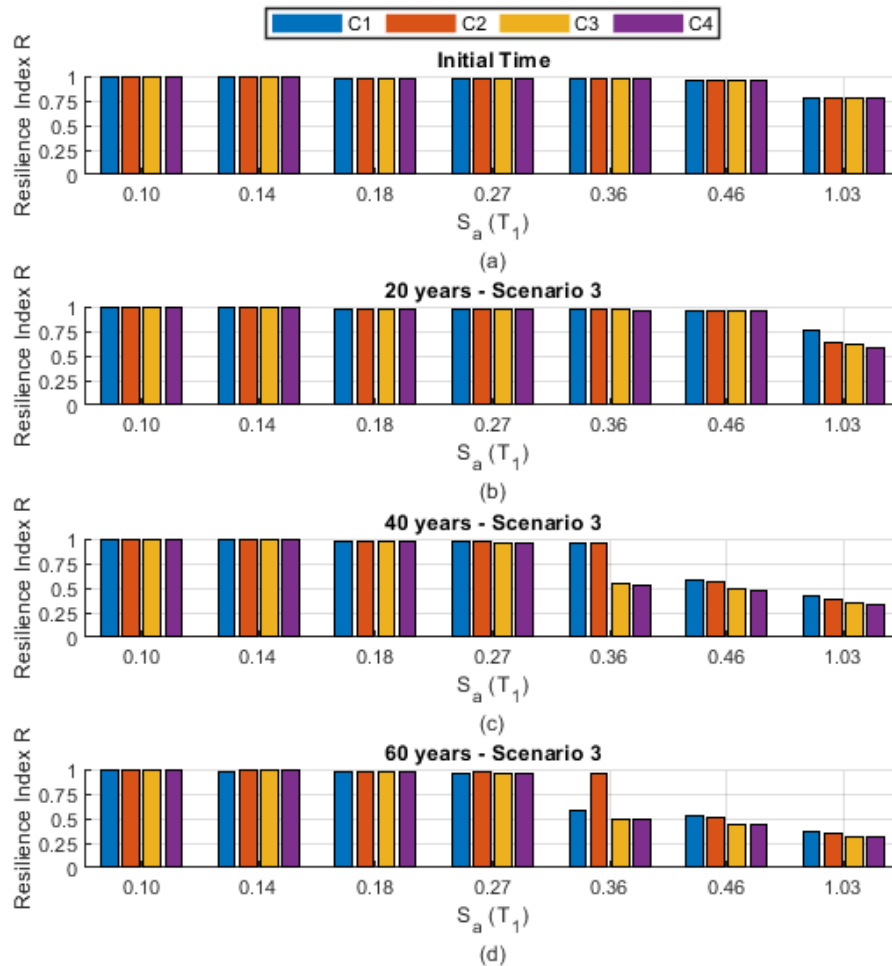


Figure 7.7.  $R$  values for selected IM levels at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years. Scenario 3.

Additional insights can be obtained by looking at the Seismic Resilience Maps for each exposure scenario and structural configuration (Figure 7.8-Figure 7.10). The results confirm that the effects of Scenario 1 on Seismic Resilience are negligible, as indicated by almost constant  $R$  values for a given IM level over time. Scenario 2 is characterized by severe reduction of  $R$  values for higher IM levels after 40 years. In this sense, C4 is more affected by deterioration with respect to the other configurations, with comparatively lower  $R$  values for each IM level when corrosion is present. As expected, the effects of deterioration on Seismic Resilience for Scenario 3 are more severe than for the other exposure scenarios. Given the severity of the environmental exposure, the results obtained for each configuration are similar, with C3 and C4 slightly more affected than C1 and C2.

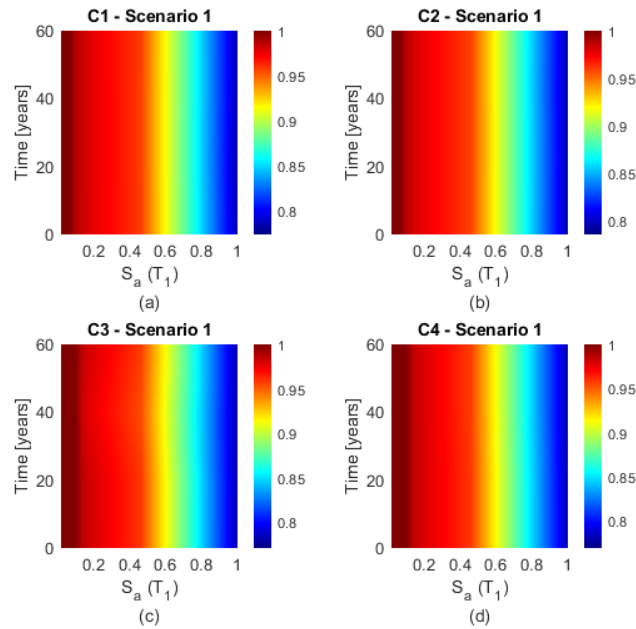


Figure 7.8. (a-d) Resilience maps for C1-C4, Scenario 1.

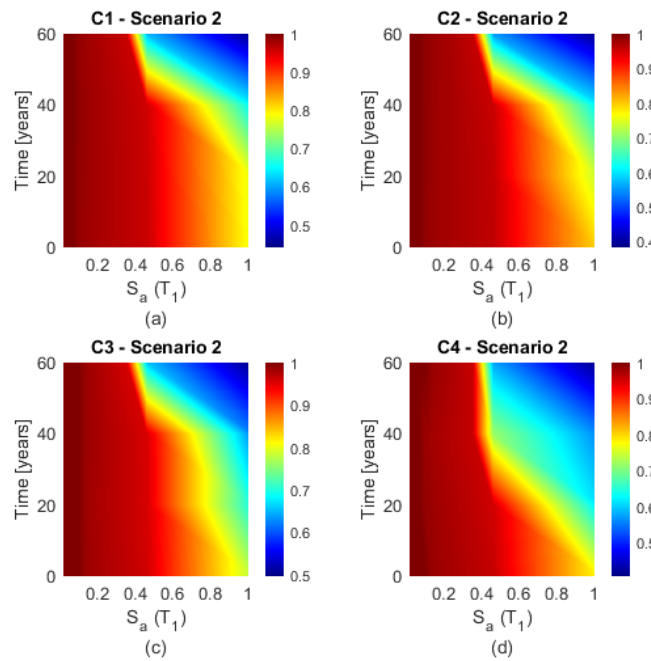


Figure 7.9. (a-d) Resilience maps for C1-C4, Scenario 2.

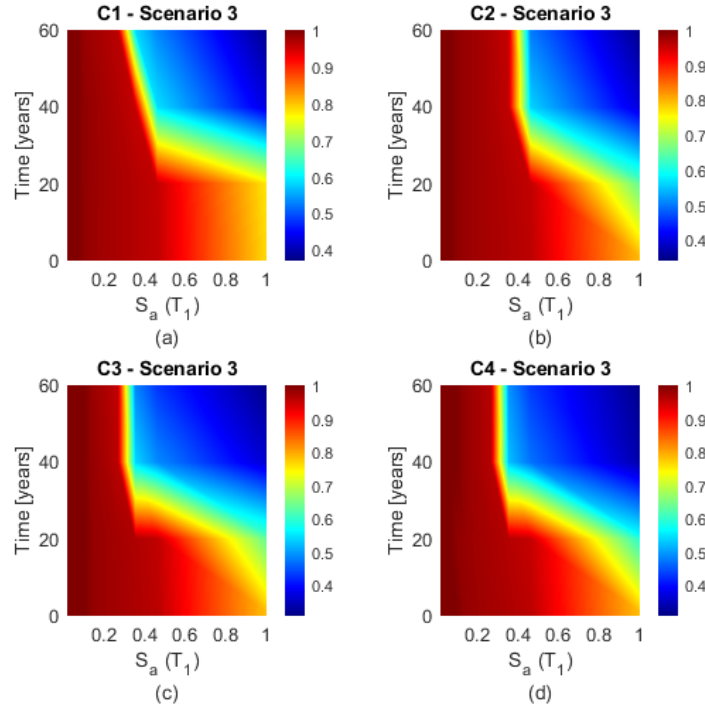


Figure 7.10. (a-d) Resilience maps for C1-C4, Scenario 3.

#### 7.4.3 Equivalent Downtime

Finally, scenario-based long term Seismic Resilience assessment for each configuration is carried out in terms of  $T_d$ . The value has been computed for IM levels characterized by  $T_R$  ranging from 250 to 100000 years. The results have been reported separately for each configuration in Table 7.9-Table 7.12, where  $T_R$  is expressed in years,  $S_a(T_1)$  in g and  $T_d$  in days.

As found in the previous sections, the initial Seismic Resilience of the different configurations is comparable. This result has been confirmed also in terms of  $T_d$ , which is directly proportional to the IM level. This result is expected and is physically represented by higher damage levels expected for higher IM levels. As previously observed, the effects of Scenario 1 exposure are negligible, with almost constant  $T_d$  values over time for a given IM level. This can be explained by considering that unbolted neoprene pad bearings play a significant role in the seismic response of the structure and the deterioration level associated with Scenario 1 does not lead to the piers dominating the behavior of the bridge.



**Table 7.9. R and  $T_d$  values for selected IM levels in each scenario at initial time, 20 years, 40 years and 60 years. Configuration 1.**

| Configuration 1 |                    |      | Initial |       | 20 years |       | 40 years |       | 60 years |       |
|-----------------|--------------------|------|---------|-------|----------|-------|----------|-------|----------|-------|
| $T_R$           | $S_a$<br>( $T_1$ ) | Case | R       | $T_d$ | R        | $T_d$ | R        | $T_d$ | R        | $T_d$ |
| 250             | 0.101              | 1    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.99     | 1     |
|                 |                    | 2    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.99     | 1     |
|                 |                    | 3    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.98     | 2     |
| 500             | 0.137              | 1    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 2     |
|                 |                    | 2    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 2     |
|                 |                    | 3    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 3     |
| 1000            | 0.185              | 1    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.98     | 3     |
|                 |                    | 2    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.98     | 3     |
|                 |                    | 3    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.97     | 4     |
| 2500            | 0.269              | 1    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.97     | 4     |
|                 |                    | 2    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.97     | 4     |
|                 |                    | 3    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.96     | 5     |
| 5000            | 0.355              | 1    | 0.97    | 5     | 0.97     | 5     | 0.97     | 5     | 0.97     | 5     |
|                 |                    | 2    | 0.97    | 5     | 0.97     | 5     | 0.97     | 5     | 0.96     | 6     |
|                 |                    | 3    | 0.97    | 5     | 0.97     | 5     | 0.96     | 6     | 0.58     | 207   |
| 10000           | 0.464              | 1    | 0.96    | 6     | 0.96     | 6     | 0.96     | 6     | 0.96     | 6     |
|                 |                    | 2    | 0.96    | 6     | 0.96     | 6     | 0.96     | 7     | 0.61     | 194   |
|                 |                    | 3    | 0.96    | 6     | 0.96     | 6     | 0.59     | 206   | 0.53     | 234   |
| 100000          | 1.031              | 1    | 0.78    | 111   | 0.77     | 112   | 0.77     | 112   | 0.77     | 112   |
|                 |                    | 2    | 0.78    | 111   | 0.77     | 114   | 0.64     | 180   | 0.44     | 279   |
|                 |                    | 3    | 0.78    | 111   | 0.77     | 116   | 0.42     | 289   | 0.37     | 314   |

Table 7.10.  $R$  and  $T_d$  values for selected IM levels in each scenario at initial time, 20 years, 40 years and 60 years. Configuration 2.

| Configuration 2 |                    |      | Initial |       | 20 years |       | 40 years |       | 60 years |       |
|-----------------|--------------------|------|---------|-------|----------|-------|----------|-------|----------|-------|
| $T_R$           | $S_a$<br>( $T_i$ ) | Case | $R$     | $T_d$ | $R$      | $T_d$ | $R$      | $T_d$ | $R$      | $T_d$ |
| 250             | 0.101              | 1    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.99     | 1     |
|                 |                    | 2    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.99     | 1     |
|                 |                    | 3    | 0.99    | 1     | 0.99     | 1     | 0.99     | 1     | 0.99     | 1     |
| 500             | 0.137              | 1    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 2     |
|                 |                    | 2    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 2     |
|                 |                    | 3    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 2     |
| 1000            | 0.185              | 1    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.98     | 3     |
|                 |                    | 2    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.98     | 3     |
|                 |                    | 3    | 0.98    | 3     | 0.98     | 3     | 0.98     | 3     | 0.98     | 3     |
| 2500            | 0.269              | 1    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.97     | 4     |
|                 |                    | 2    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.97     | 4     |
|                 |                    | 3    | 0.97    | 4     | 0.97     | 4     | 0.97     | 4     | 0.97     | 5     |
| 5000            | 0.355              | 1    | 0.97    | 5     | 0.97     | 5     | 0.97     | 5     | 0.97     | 5     |
|                 |                    | 2    | 0.97    | 5     | 0.97     | 5     | 0.96     | 5     | 0.96     | 6     |
|                 |                    | 3    | 0.97    | 5     | 0.97     | 5     | 0.96     | 6     | 0.95     | 7     |
| 10000           | 0.464              | 1    | 0.96    | 6     | 0.96     | 6     | 0.96     | 6     | 0.96     | 6     |
|                 |                    | 2    | 0.96    | 6     | 0.96     | 6     | 0.95     | 7     | 0.55     | 223   |
|                 |                    | 3    | 0.96    | 6     | 0.96     | 7     | 0.56     | 219   | 0.50     | 248   |
| 100000          | 1.031              | 1    | 0.79    | 106   | 0.79     | 106   | 0.79     | 106   | 0.79     | 106   |
|                 |                    | 2    | 0.79    | 106   | 0.73     | 136   | 0.59     | 202   | 0.38     | 308   |
|                 |                    | 3    | 0.79    | 106   | 0.63     | 182   | 0.39     | 304   | 0.34     | 328   |

Table 7.11.  $R$  and  $T_d$  values for selected IM levels in each scenario at initial time, 20 years, 40 years and 60 years. Configuration 3.

| Configuration 3 |                    |      | Initial |       | 20 years |       | 40 years |       | 60 years |       |
|-----------------|--------------------|------|---------|-------|----------|-------|----------|-------|----------|-------|
| $T_R$           | $S_a$<br>( $T_1$ ) | Case | $R$     | $T_d$ | $R$      | $T_d$ | $R$      | $T_d$ | $R$      | $T_d$ |
| 250             | 0.101              | 1    | 1.00    | 0     | 1.00     | 0     | 1.00     | 0     | 1.00     | 0     |
|                 |                    | 2    | 1.00    | 0     | 1.00     | 0     | 1.00     | 0     | 1.00     | 0     |
|                 |                    | 3    | 1.00    | 0     | 1.00     | 0     | 1.00     | 0     | 1.00     | 0     |
| 500             | 0.137              | 1    | 0.99    | 1     | 0.99     | 1     | 0.98     | 2     | 0.99     | 2     |
|                 |                    | 2    | 0.99    | 1     | 0.98     | 2     | 0.99     | 1     | 0.99     | 1     |
|                 |                    | 3    | 0.99    | 1     | 0.99     | 1     | 0.98     | 2     | 0.98     | 2     |
| 1000            | 0.185              | 1    | 0.98    | 2     | 0.98     | 2     | 0.98     | 3     | 0.98     | 2     |
|                 |                    | 2    | 0.98    | 2     | 0.98     | 2     | 0.98     | 2     | 0.98     | 3     |
|                 |                    | 3    | 0.98    | 2     | 0.98     | 2     | 0.98     | 3     | 0.97     | 4     |
| 2500            | 0.269              | 1    | 0.97    | 3     | 0.97     | 3     | 0.97     | 4     | 0.97     | 3     |
|                 |                    | 2    | 0.97    | 3     | 0.97     | 4     | 0.97     | 3     | 0.97     | 4     |
|                 |                    | 3    | 0.97    | 3     | 0.97     | 3     | 0.96     | 6     | 0.95     | 7     |
| 5000            | 0.355              | 1    | 0.97    | 4     | 0.97     | 4     | 0.96     | 5     | 0.97     | 5     |
|                 |                    | 2    | 0.97    | 4     | 0.97     | 5     | 0.97     | 5     | 0.96     | 6     |
|                 |                    | 3    | 0.97    | 4     | 0.97     | 5     | 0.54     | 229   | 0.48     | 257   |
| 10000           | 0.464              | 1    | 0.96    | 6     | 0.96     | 6     | 0.95     | 7     | 0.96     | 6     |
|                 |                    | 2    | 0.96    | 6     | 0.96     | 7     | 0.95     | 7     | 0.66     | 171   |
|                 |                    | 3    | 0.96    | 6     | 0.95     | 7     | 0.49     | 255   | 0.43     | 285   |
| 100000          | 1.031              | 1    | 0.77    | 113   | 0.77     | 113   | 0.77     | 113   | 0.77     | 113   |
|                 |                    | 2    | 0.77    | 113   | 0.69     | 156   | 0.62     | 191   | 0.50     | 251   |
|                 |                    | 3    | 0.77    | 113   | 0.62     | 191   | 0.36     | 321   | 0.31     | 345   |

Table 7.12. R and T<sub>d</sub> values for selected IM levels in each scenario at initial time, 20 years, 40 years and 60 years. Configuration 4.

| Configuration 4 |                                     |      | Initial |                | 20 years |                | 40 years |                | 60 years |                |
|-----------------|-------------------------------------|------|---------|----------------|----------|----------------|----------|----------------|----------|----------------|
| T <sub>R</sub>  | S <sub>a</sub><br>(T <sub>1</sub> ) | Case | R       | T <sub>d</sub> | R        | T <sub>d</sub> | R        | T <sub>d</sub> | R        | T <sub>d</sub> |
| 250             | 0.101                               | 1    | 1.00    | 0              | 1.00     | 0              | 1.00     | 0              | 1.00     | 0              |
|                 |                                     | 2    | 1.00    | 0              | 0.99     | 1              | 0.98     | 2              | 1.00     | 0              |
|                 |                                     | 3    | 1.00    | 0              | 0.99     | 1              | 1.00     | 0              | 1.00     | 0              |
| 500             | 0.137                               | 1    | 0.99    | 1              | 0.99     | 1              | 0.99     | 1              | 0.99     | 1              |
|                 |                                     | 2    | 0.99    | 1              | 0.98     | 2              | 0.98     | 3              | 0.99     | 2              |
|                 |                                     | 3    | 0.99    | 1              | 0.98     | 2              | 0.98     | 2              | 0.98     | 2              |
| 1000            | 0.185                               | 1    | 0.98    | 2              | 0.98     | 2              | 0.98     | 2              | 0.98     | 2              |
|                 |                                     | 2    | 0.98    | 2              | 0.98     | 3              | 0.97     | 4              | 0.98     | 2              |
|                 |                                     | 3    | 0.98    | 2              | 0.98     | 3              | 0.98     | 3              | 0.98     | 3              |
| 2500            | 0.269                               | 1    | 0.97    | 3              | 0.97     | 3              | 0.97     | 3              | 0.97     | 3              |
|                 |                                     | 2    | 0.97    | 3              | 0.97     | 4              | 0.97     | 5              | 0.97     | 4              |
|                 |                                     | 3    | 0.97    | 3              | 0.97     | 4              | 0.96     | 6              | 0.96     | 6              |
| 5000            | 0.355                               | 1    | 0.97    | 5              | 0.97     | 5              | 0.97     | 5              | 0.97     | 5              |
|                 |                                     | 2    | 0.97    | 5              | 0.96     | 5              | 0.96     | 6              | 0.96     | 6              |
|                 |                                     | 3    | 0.97    | 5              | 0.96     | 5              | 0.52     | 237            | 0.49     | 253            |
| 10000           | 0.464                               | 1    | 0.96    | 6              | 0.96     | 6              | 0.96     | 6              | 0.96     | 6              |
|                 |                                     | 2    | 0.96    | 6              | 0.95     | 7              | 0.72     | 140            | 0.58     | 211            |
|                 |                                     | 3    | 0.96    | 6              | 0.95     | 7              | 0.47     | 264            | 0.43     | 283            |
| 100000          | 1.031                               | 1    | 0.77    | 113            | 0.77     | 114            | 0.77     | 114            | 0.77     | 114            |
|                 |                                     | 2    | 0.77    | 113            | 0.61     | 196            | 0.56     | 221            | 0.41     | 296            |
|                 |                                     | 3    | 0.77    | 113            | 0.57     | 213            | 0.33     | 334            | 0.31     | 345            |

Structural deterioration effects are not negligible only for IM levels associated with  $T_R$  of 5000 years and higher, depending on structural configuration and scenario. As expected,  $T_d$  increases, for a fixed IM level, as time passes and structural deterioration level increases. While the results obtained in the previous sections are confirmed in terms of  $T_d$  values, it is worth pointing out that this quantity represents the natural complement of R and is more physically understandable by stakeholders and managers, representing a useful tool for post-event management of the structure.

A final consideration is related to the IM levels for which structural deterioration effects on Seismic Resilience are relevant. It has been observed that each configuration is associated with high time-variant Seismic Resilience for IM levels with  $T_R$  up to 2500 years, which represents rare seismic events. In this sense, each configuration is expected to respond satisfactorily to seismic events and to be reopened in a short time given the low level of expected damage. However, deterioration phenomena should not be underestimated given their significant effect on expected Seismic Resilience for higher IM levels (associated with very rare events with very long  $T_R$ ). This investigation indicates how Seismic Resilience is a concept that could be used in prioritization schemes to optimize the allocation of resources for infrastructure networks management, guiding retrofit interventions and emergency management planning. Future research efforts should be dedicated to the implementation of Seismic Resilience as a specific design or retrofit target for bridges in regions with relevant seismicity, integrating it with economic considerations oriented towards the identification of optimal management strategies.

## 7.5 CONCLUSIONS

This chapter addressed the long-term Seismic Resilience assessment of existing bridge structure configurations. The framework introduced in Chapter 6 has been applied to four bridge configurations inspired by a real case study bridge. Three progressively more severe chloride exposure scenarios have been considered for the long-term Seismic Resilience assessment.

The results have been first compared in terms of fragility curves, highlighting how Scenario 1 was associated with almost negligible effects, while Scenario 2 and Scenario 3 led to significant increase of seismic vulnerability of the four configurations over time. The increase of seismic vulnerability was found to be more pronounced passing from Scenario 1 to Scenario 3, with a maximum reduction of median IM value of 53.07%, observed at 60 years for C4. The increase of seismic vulnerability of the configurations affected significantly the long-term Seismic Resilience assessment. This has been explained both mathematically, considering that increased seismic vulnerability leads to higher losses and lower functionality over the recovery phase, and physically, given that increased vulnerability is associated with higher damage level, repair cost and repair time after a seismic event.

Seismic Resilience has been initially compared in terms of mean  $R$  values over time for IM levels 4-10. The results highlighted that the initial seismic performance of each configuration was comparable. As expected, Seismic Resilience was inversely proportional to the IM level.  $R$  values were found to be progressively lower passing from Scenario 1 to Scenario 3. Deterioration effects were found to become more evident for progressively lower IM levels as time passed, as a consequence of increased vulnerability and shift of

$DS_P$ , but they were always negligible up to IM levels associated with  $T_R$  lower than 5000 years. Seismic Resilience maps have been presented for each configuration and exposure scenario, providing further confirmation of the results previously observed and identifying C3 and C4 as the configurations most affected by deterioration.

Finally, the results have been compared in terms of  $T_d$ , for IM levels characterized by  $T_R$  ranging from 250 to 100000 years. The results observed in terms of  $R$  have been confirmed, with deterioration effects that are not negligible only for IM levels associated with  $T_R$  of 5000 years and higher, depending on structural configuration and scenario. Each configuration is associated with high time-variant Seismic Resilience for IM levels with  $T_R$  up to 2500 years, representing rare seismic events, indicating satisfactory response to seismic events and short downtime. Deterioration effects should not be underestimated given their effects on expected Seismic Resilience for very rare IM levels.

In conclusion, Seismic Resilience is a concept that could be used in prioritization schemes to optimize resource allocation for infrastructure networks management, guiding retrofit interventions and emergency management planning. Future research should be oriented towards the implementation of Seismic Resilience as a specific design or retrofit target for bridges in regions with relevant seismicity, integrating it with economic considerations to identify rigorously optimal management, inspection and repair strategies.

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## **8. EXTENSION OF DIRECT DISPLACEMENT BASED ASSESSMENT PROCEDURES FOR LIFE CYCLE ASSESSMENT AND UNCERTAINTY ANALYSIS OF BRIDGE STRUCTURES**

### **8.1 INTRODUCTION**

Bridge structures are often exposed to harsh environmental conditions that deteriorate Reinforced Concrete (RC) members, affecting the structural response over time. It is therefore clear that deterioration phenomena must be considered in the framework of seismic risk assessment of structures and networks, and a Life-Cycle-oriented approach is necessary. Given the high computational onus associated with performing Nonlinear Time History Analysis (NLTHA) on a large scale in a Life Cycle assessment framework, simplified methodologies based on equivalent static approaches are appealing. The principles of Direct Displacement Based Design (DDBD), relying on the definition of an equivalent SDOF system representing the entire structure, have been formalized in the early 2000's. A significant amount of research has been carried out over the years to extend the DDBD principles to the assessment of existing structures and to align them with the principles of Performance Based Earthquake Engineering (PBEE).

DDBD was conceived as design method, simplifying complex problems such as the design of seismic isolation systems for irregular bridge structures [1]. The main objective of the methodology, aligned with the first conception of PBEE, was to design structures in order to achieve predictable levels of performance under defined levels of seismic input, explicitly controlling the structural response in the design phase [2]. The method was further developed to include different isolation systems and dissipation devices and validated against NLTHAs [3]. Extensive research effort was dedicated to the development of inelastic displacement patterns for bridge structures, highlighting the importance of boundary conditions and relative stiffness of superstructure and substructure [4]. DDBD principles have been later applied to the Direct Displacement Based Assessment (DDBA) of bridge structures, as in Cardone et al. [5], demonstrating promising results towards the integration of DDBD in the PBEE framework. Significant challenges have to be faced trying to reach that goal, as discussed by De Luca et al. [6] who investigated the bias of capacity curve fitting procedures and proposed an optimal piecewise linear fitting method for equivalent SDOF analysis. Welch et al. [7] developed a simplified probabilistic loss

assessment methodology based on the DDBD principles, finding results fairly similar to the PEER PBEE methodology. O'Reilly and Calvi [8] investigated the risk-oriented extension of DDBA, comparing a simplified methodology to the refined approach and finding good agreement between the two assessment procedures. The research carried out by Perdomo and Monteiro [9] systematically investigated and extended existing displacement-based formulations to estimate expected annual losses of RC bridges exposed to transverse excitation. The results revealed that simplified approaches tend to overestimate losses with respect to the fully probabilistic framework and are sensitive to the definition of the collapse fragility curve. A recent study [10] discussed the capability of DDBD of producing risk-consistent designs, pointing out the limitations of intensity-based design methods not accounting for uncertainty to maintain risk consistency.

The present chapter consists of a collection of state-of-the-art approaches for the description of the seismic response of bridge structures and of the subsequent integration of the obtained Engineering Demand Parameter (EDP) distributions in the PBEE framework, aiming at reducing the computational burden associated with Life Cycle assessment of bridge structures. The approach is validated through comparison with results obtained using NLTHAs and taken as benchmark. Two different bridge configurations subjected to deterioration are considered for validation. Different environmental exposure conditions are investigated in order to identify possible deviations from the exact results due to the level of deterioration. Given the highly reduced computational cost of the approach, an application to uncertainty analysis in the life cycle assessment is carried out, considering a probabilistic characterization of material properties and deterioration evolution, and the results are discussed in terms of fragility curves, seismic risk, and Expected Annual Loss Ratio (EALR). The results are scrutinized, and the possible applications of the methodology are discussed.

## 8.2 METHODOLOGY

The methodology, consisting of a collection of state-of-the-art approaches for the simplified assessment of RC bridges implemented in in-house MATLAB routines, is organized into several sub-steps, represented in Figure 8.1. Each sub-step will be discussed in the following sections.

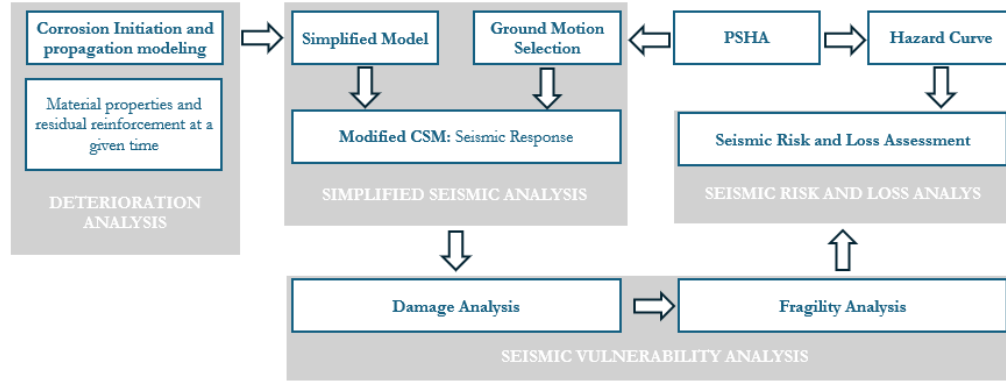


Figure 8.1. Flowchart of the methodology.

### 8.2.1 Simplified Characterization of Bridge Components

In the DDBA framework, the seismic response of RC bridges is represented by an equivalent SDOF oscillator reproducing the dynamic characteristics of the entire structure. In order to define the equivalent SDOF system, the first step is characterizing the simplified capacity curves of bridge components. The definition of capacity curves depends on the analysis direction, and several assumptions are necessary in this phase. It is assumed that the superstructure behaves linearly under seismic excitation, and nonlinear behavior is assumed to take place in correspondence of bearings, piers and abutments, consistently with the common practice in the seismic analysis of bridge structures [11]. The effects of foundational flexibility are neglected. While a few studies [12] discussed the inclusion of Soil Structure Interaction (SSI) in the DDBA framework, research is still necessary in order to obtain a generalized formulation. It is also worth noting that existing approaches to include SSI effects, such as [12], are more suitable for first-level screening of bridges with potentially significant SSI effects. Additionally, the conservatism in foundation design justifies the fixed base assumption made in the present investigation [13]. Lastly, each superstructure-substructure assembly is modeled as an equivalent SDOF. The capacity curve of the subsystem is obtained by assembling that of its components, and the corresponding tributary mass is lumped on top of it.

The capacity curves of RC piers can be obtained through the approach proposed in [14], which is based on bilinearized moment-curvature analysis [15]. Yielding displacement  $\Delta_y$  and ultimate displacement  $\Delta_u$  are computed as per Equations (8.1)-(8.2) [16]:

$$(8.1) \quad \Delta_y = \frac{\phi_y (L + C_f L_{sp})^2}{3}$$

$$(8.2) \quad \Delta_u = \Delta_y + (\phi_u - \phi_y) L_p C_f (L - 0.5 L_p)$$

where  $\varphi_y$  is the yield curvature,  $\varphi_u$  is the ultimate curvature,  $L$  is the distance between base of the pier and point of counterflexure of the deformed shape,  $C_f$  is a coefficient equal to 1 or 2 for single or double bending piers respectively,  $L_{sp}$  is the strain penetration length and  $L_p$  is the plastic hinge length. According to [14],  $L_{sp}$  is taken as  $0.022f_{yt}d_{bl}$ , where  $f_{yt}$  is the yield strength of longitudinal reinforcement and  $d_{bl}$  is the minimum diameter of longitudinal bars.  $L_p$  is defined as  $0.08L + 0.022f_{yt}d_{bl}$  according to the formulation proposed in [17]. It is worth noting that, besides the flexural response, additional potential failure modes (such as shear failure [18]) must be taken into account in the pre-processing phase, modifying the capacity curve accordingly.

The identification of the capacity curve for single column piers can be carried out through the described approach, while for multi-column piers the capacity curve can be obtained by aggregating the force-displacement relationships of single columns working in parallel, assuming single or double bending conditions depending on the direction of analysis.

In case the deformation of bearing devices is negligible, they can be assumed to behave as fixed bodies, considering nonlinearities only in the substructure. In case their deformation is not negligible, as in the case of existing bridges presenting non-seismically designed devices or seismically isolated bridges, the specific capacity curves of bearing devices must be defined, and the equivalent curves of each superstructure-to-substructure connection system have to be defined by aggregating the laws of individual devices assuming they work in parallel. A comprehensive set of capacity curves for different types of bearing systems can be found in [16]. Shear keys can be present on the structure, in order to provide transverse support in case of earthquakes. Based on the numerical and experimental investigation carried out by Megally et al. [19], capacity curves for shear keys have been proposed [19].

The equivalent SDOF system representing the assembly of substructure and connection system can be defined by aggregating the curves of the subsystems assuming they work in series.

Regarding the abutments, rigid behavior can be assumed for the transverse analysis, while in the longitudinal direction it is necessary to consider the interaction between joints, bearing devices, backwall and backfill soil. Prior to the gap closure, inertia force is transmitted only through the bearing devices, while after gap closure the deck starts to push on the backwall, interacting also with the backfill soil. More details on the abutments characterization can be found in References [16][20][21].

### 8.2.2 DDBA based on Modal Analysis

The DDBA procedure based on Modal Analysis was first proposed by Şadan et al. [22] and has been successfully applied for the analysis of different bridge configurations [16][23]. The main steps of the procedure are summarized in Figure 8.2. It is worth remarking that the procedure is useful to identify the displaced shape of the bridge in transverse direction, while in longitudinal direction it can be assumed that each SDOF subassembly works in parallel. A control node, which will control the identification of the displaced shape, has to be identified. Generally, taking the top node of a pier close to the central part of the deck is a good choice. The structure is represented as a series of elastic beams, representing the deck, over spring supports representing the equivalent SDOF systems identified before. The mass of each SDOF system is computed based on their tributary length.

After identifying the control node, it is necessary to provide an initial guess of the displaced shape, scaled such that the displacement of the control node is equal to the target displacement  $\Delta_i$ . The literature includes a number of displaced shapes that could be used for the initial guess [14][16]. Based on the displaced shape, the shear force  $V_i$  in the  $i$ -th subassembly can be computed, and the corresponding secant stiffness  $k_{sec,i}$  can be found as per Equation (8.3):

$$(8.3) \quad k_{sec,i} = \frac{V_i}{\Delta_i}$$

where  $\Delta_i$  is the displacement of the  $i$ -th subassembly. The values of  $k_{sec,i}$  are assigned to the elements, and modal analysis is performed, obtaining the first transverse modal shape  $\Phi_i$  and its participating mass  $M_i^*$ . The modal shape obtained through the eigenvalue analysis is then scaled as per Equation (8.4):

$$(8.4) \quad \Delta'_i = \Delta_t \frac{\Phi_i}{\Phi_c}$$

where  $\Delta'_i$  is the new displacement profile,  $\Phi_i$  is the vector with the modal displacements and  $\Phi_c$  is the modal displacement of the control node.  $\Delta'_i$  is then compared with the previous displaced shape  $\Delta_{i-1}$ . In case of convergence,  $\Delta'_i$  is taken as the displaced shape of the bridge. A tolerance of 0.001 m is recommended to achieve a fast convergence. In case of no-convergence, Equation (8.3) is applied and a new eigenvalue analysis is performed, until the computed shape stabilizes.

Higher mode effects should be taken into account in the process if  $M_i^*$  is less than 70%. The displaced shape will be computed including a number of modes such as the cumulative participating mass exceeds 90%. The displaced shape of the  $j$ -th mode  $\Delta_{ij}$  can be computed as per Equation (8.5):

$$(8.5) \quad \Delta_{i,j} = \Phi_{i,j} \Gamma_j S_d(T_j)$$

where  $\Gamma_j$  is the modal participation factor,  $T_j$  is the period of vibration of the j-th mode and  $S_d(T_j)$  is the displacement demand computed from a 5% damped response spectrum. The final displaced shape is obtained through combination of all the relevant mode shapes.

Once the transverse displaced shape of the bridge has been established, it is possible to define the equivalent SDOF system representing the bridge structure in correspondence of the specific target point. The equivalent SDOF system is characterized in terms of effective displacement  $\Delta_{eff}$ , effective mass  $m_{eff}$ , effective period  $T_{eff}$  and effective damping  $\xi_{eff}$ , as per Equations (8.6)-(8.9):

$$(8.6) \quad \Delta_{eff} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i}$$

$$(8.7) \quad m_{eff} = \frac{\sum m_i \Delta_i}{\Delta_{eff}}$$

$$(8.8) \quad T_{eff} = \frac{\Delta_{eff}}{S_{a,eff}}$$

$$(8.9) \quad \xi_{eff} = \frac{0.05 \Delta_{ab,1} V_{ab,1} + 0.05 \Delta_{ab,2} V_{ab,2} + 0.02 \Delta_{eff} (V_{ab,1} + V_{ab,2}) + \sum \xi_{eqpier,k} \Delta_{eqpier,k} V_{pier,k}}{\Delta_{ab,1} V_{ab,1} + \Delta_{ab,2} V_{ab,2} + \Delta_{eff} (V_{ab,1} + V_{ab,2}) + \sum \Delta_{eqpier,k} V_{pier,k}}$$

where  $m_i$  is the i-th lumped mass,  $\Delta_i$  is the displacement of the i-th lumped mass,  $S_{a,eff}$  is the spectral acceleration of the SDOF system corresponding to the displaced shape,  $V_{ab}$  is the shear in one abutment,  $\Delta_{ab}$  is the displacement of one abutment,  $\xi_{eqpier,k}$  is the equivalent viscous damping of the k-th pier-bearings subassembly,  $\Delta_{eqpier,k}$  is the equivalent displacement of the k-th pier-bearings subassembly and  $V_{pier,k}$  is the shear in the k-th pier-bearing system subassembly.  $\xi_{eqpier,k}$  can be computed according to Equations (8.10)-(8.11):

$$(8.10) \quad \xi_{eqpier,k} = \frac{\Delta_{pier} \xi_{pier} + \Delta_{bearings} \xi_{bearings}}{\Delta_{eqpier,k}}$$

$$(8.11) \quad \xi_{pier} = 0.05 + \frac{0.444(\mu_k - 1)}{\mu_k \pi}$$

where  $\Delta_{pier}$  is the displacement of the pier,  $\xi_{pier}$  is the ductility-based equivalent viscous damping of the pier [14],  $\Delta_{bearings}$  is the displacement of the bearing system,  $\xi_{bearings}$  is the equivalent viscous damping provided by the bearings and  $\mu_k$  is the ductility demand on the pier.



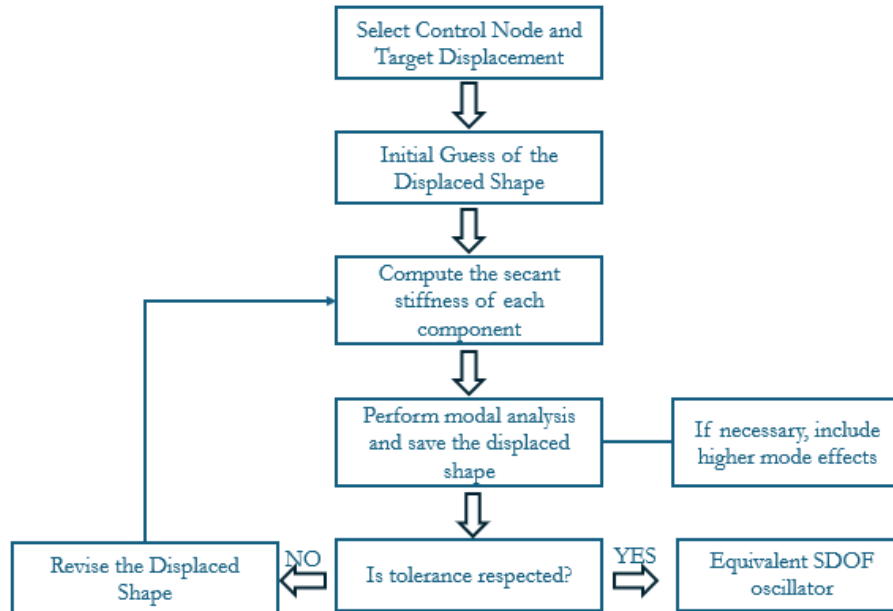


Figure 8.2. DDBA based on modal analysis.

### 8.2.3 Displacement Based Pseudo Pushover Curve

Based on the concept of the Modal DDBA, Gentile et al. [23] proposed an iterative application of the method to derive the capacity curve of a bridge structure. The Modal DDBA procedure is repeated for increasing levels of displacement, providing to the analyst information on the behavior of the bridge over different displacement levels. For each step of the analysis, the properties describing the equivalent SDOF structure (Equations (8.6)-(8.11)) and the Engineering Demand Parameters (EDPs) for each structural element are recorded. The capacity curve is represented as the base shear vs  $\Delta_{eff}$  plot (Figure 8.3). The capacity curve is therefore based on a series of linear analysis with members described using their secant stiffness at the corresponding level of displacement. It is worth remarking that each step of the analysis refers to an independent displacement profile and that the equivalent SDOF system is compatible with the secant stiffness of the structural members, essentially making the algorithm adaptive. The method has been validated through comparison with an extensive set of NLTHAs. The interested reader may refer to [23] for more details on the algorithm and on its validation.

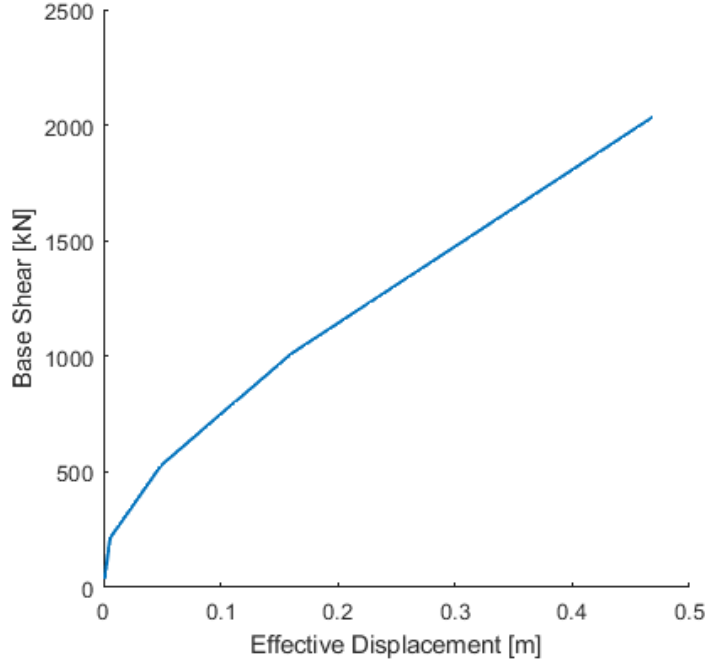


Figure 8.3. Displacement Based Pseudo Pushover Curve.

#### 8.2.4 Seismic Response Estimation

Once that the Displacement Based Pseudo Pushover (DBPP) curve has been obtained, the seismic response for a given set of ground motions can be estimated through one of the available Acceleration-Displacement Response Spectrum (ADRS) methods. In order to obtain a refined description of the record-to-record variability, the method proposed by Nettis et al. [25] is used. The method is an extension of the original Capacity Spectrum Method proposed by Freeman in 1998 [26] employing response spectra of natural ground motions. Given the possibility of multiple intersections between bridge capacity curve and natural ground motion spectra [27], the algorithm includes indications to select the correct Performance Point for a given record. Starting from the DBPP Curve, knowing that each displacement level  $\Delta_i$  is associated with an equivalent SDOF system described by Equations (8.6)-(8.11), the corresponding spectral reduction factor  $\eta$  can be computed according to Equation (8.12):

$$(8.12) \quad \eta = \sqrt{\frac{0.07}{0.02 + \xi_{eff}}}; \quad \eta > 0.5$$

For each of the equivalent SDOF systems included in the DBPP curve, the elastic acceleration-displacement demand for a given ground motion is computed at  $T_{eff}$

considering a 5% equivalent viscous damping. The overdamped acceleration-displacement demand is computed by multiplying the elastic demand for  $\eta$ . The resulting acceleration-displacement demand spectra is referred to as “variable damping” due to the different characteristics associated with the SDOF oscillators included in the DBPP curve. The possible Performance Points are the intersections between the capacity curve and the variable damping spectra. In case of no intersection, the bridge is assumed to not be able to sustain the ground motion, and a Complete Damage condition is identified. Several approaches have been tested to identify the correct Performance Point in case of multiple intersections, but the approaches based on the average Spectral Displacement over a range of periods  $AvgSd_k$  and on the selection of the first intersection as the Performance Point performed better than the others. In the present investigation, the  $AvgSd_k$ -based approach has been used in case of multiple intersections. More details on the algorithm can be found in [25]. The approach has been validated through a comprehensive series of NLTHAs, confirming the capabilities of the algorithm for the estimation of seismic response of structures and of median and dispersion values of fragility curves [25][28].

#### 8.2.5 Seismic Hazard

The existing research on simplified approaches for seismic vulnerability assessment explored the use of stripe-based analysis [29]. The approach used in the present research is inspired by the current practice and by the principle of Multiple Stripe Analysis [30]. 10 seismic hazard levels with return period  $T_R$  of 10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 50000 years are considered. According to previous research on the seismic assessment of RC bridges [31][32], the spectral acceleration corresponding to the first mode period of the bridge  $Sa(T_1)$  is selected as Intensity Measure (IM). Deaggregation of the seismic hazard [33] is performed at each IM level to characterize the events contributing to hazard in terms of magnitude  $M$ , distance to source  $R$  and epsilon parameter  $\varepsilon$  [34]. Deaggregation data and site-specific hazard curve can be obtained using a Probabilistic Seismic Hazard Analysis (PSHA) software such as REASSESS [35]. Hazard consistent records can be selected for each IM level based on the PSHA results using the Conditional Spectrum method [36][37]. It is worth noting that selecting the input motions matching mean and variance of the Conditional Spectrum, based on the site-specific PSHA results, allows for explicitly considering the record-to-record variability in the analysis.

#### 8.2.6 Damage States

Four different Damage States (DS) have been considered in the present work, namely Slight Damage, Moderate Damage, Extensive Damage and Complete Damage. The exceedance of each DS is computed based on the ratio between the displacement demand and the DS thresholds for each component.

For each record, the maximum value of the demand/capacity ( $D/C$ ) ratios among the values associated with each component is considered to identify the structural performance. Note that this corresponds to assuming that the bridge is a series system. Table 8.1 describes the thresholds adopted for the analysis of the case studies considered in the present work. The interested reader may refer to [16] for more information.

**Table 8.1. Damage State Thresholds for different bridge components.**  $\Delta_{sh}$ =displacement corresponding to shear failure.  $d_{fr}$ =displacement corresponding to the onset of sliding.  $d_{pad}$ =pad dimension in the motion direction.  $d_{uns}$ =displacement corresponding to girder unsearing.  $\gamma$ =shear deformation of the bearing device.

| Component              | Slight Damage    | Moderate Damage                       | Extensive Damage                      | Complete Damage  |
|------------------------|------------------|---------------------------------------|---------------------------------------|------------------|
| Piers (flexural)       | $\Delta_y$       | $\Delta_y + 1/2(\Delta_u - \Delta_y)$ | $\Delta_y + 2/3(\Delta_u - \Delta_y)$ | $\Delta_u$       |
| Piers (shear)          | -                | -                                     | $\Delta_{sh}$                         | $1.1\Delta_{sh}$ |
| Unbolted Neoprene Pads | $d_{fr}$         | $d_{fr} + 1/3(d_{pad} - d_{fr})$      | $d_{pad}$                             | $d_{uns}$        |
| Elastomeric Bearings   | $\gamma = 150\%$ | $\gamma = 200\%$                      | $\gamma = 300\%$                      | $d_{uns}$        |

### 8.2.7 Fragility Analysis and Seismic Risk Assessment

Fragility functions, expressing the probability of exceeding a given DS conditioned on the level of IM, are computed using the Maximum Likelihood Estimation method [38]. Fragility functions are described as lognormal cumulative distribution functions, according to Equation (8.13):

$$(8.13) \quad P(F|IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta_{IM}}\right)$$

where  $P(F|IM=x)$  is the probability of exceeding a given DS conditioned on IM,  $x$  is the IM value,  $\theta$  is the median of the fragility function and  $\beta_{IM}$  is the dispersion of IM.

Seismic Risk is quantified in terms of Mean Annual Frequency of Exceedance  $\lambda_f$  of a given DS, which can be expressed as:

$$(8.14) \quad \lambda_f = \int_0^\infty P(F|IM = x) |d\lambda(IM > x)|$$

where  $|d\lambda(IM > x)|$  is the absolute value of the derivative of the site hazard curve.

### 8.2.8 Seismic Loss Analysis

Seismic Loss Analysis is carried out according to the HAZUS approach, first implemented in [39]. The damage ratios  $\mu_{D,i}$  for each DS are reported in Table 8.2. The Compound Damage Ratio  $CDR$  is computed as per Equation (8.15):

$$(8.15) \quad CDR = \sum_{i=1}^4 \mu_{D,i} \cdot P(DS = DS_i);$$

where  $P(DS=DS_i)$  is the probability of being in the  $i$ -th DS.  $CDR$  represents the expected losses normalized on the replacement cost of the structure given the IM value. Starting from this definition, the Expected Annual Loss Ratio  $EALR$  can be computed according to Equation (8.16) [40]:

$$(8.16) \quad EALR = \int_0^{\infty} CDR |d\lambda(IM > x)|$$

It is worth remarking that the HAZUS approach only includes the direct seismic losses of the structure, while the computation of indirect losses would require advanced network simulations, and it is out of the scope of this study. The HAZUS approach is a structure-level approach and generally represents an upper bound to more refined component-level approaches, producing results that are well aligned with the latter approaches [41].

Table 8.2. Damage Ratios for the DS considered in the study, according to [39].

| Damage State     | Damage Ratio                                                     |
|------------------|------------------------------------------------------------------|
| Slight Damage    | 0.03                                                             |
| Moderate Damage  | 0.08                                                             |
| Extensive Damage | 0.25                                                             |
| Complete Damage  | 1.00 (or 2.00/number of spans, for bridges with 3 or more spans) |

### 8.3 CORROSION MODEL

Life Cycle Analysis is carried out by considering the evolution of bridge structures over time according to a commonly used corrosion model [42][43]. While the present study refers to chloride-induced corrosion, the approach is readily extendible for different conditions, such as carbonation-induced corrosion. Longitudinal and transverse reinforcement sections are reduced over time according to a localized (pitting) corrosion model. The mechanical properties of steel and concrete materials are modified consistently with the corrosion level according to specific deterioration laws.

### 8.3.1 Corrosion Initiation Phase

Corrosion initiation phase, ranging from the initial time to the point of time when corrosion initiates, is modeled according to the *fib* model [42][43]. The model applies the error function solution of 1D Fick's Second Law of Diffusion to estimate time- and depth-dependent chloride concentration  $C(x,t)$  in the concrete matrix. Corrosion is assumed to start when the critical concentration of free chloride ions at the depth of the external surface of reinforcement is reached. Due to the significant uncertainty of the chloride diffusion process, a Monte Carlo simulation approach is used. The variables of the process are characterized as Random Variables (RVs), and the statistical distributions assumed for their description can be found in References [42]-[44].

### 8.3.2 Corrosion Propagation Phase

The corrosion propagation phase ranges from corrosion initiation time to the end of the service life and is taken into account in the Monte Carlo simulation. The corrosion propagation phase can be described following Equation (8.17) [45]:

$$(8.17) \quad p(t_p) = 0.0116 \cdot R \cdot t_p \cdot i_{corr}(t)$$

where  $p(t_p)$  is the maximum pit depth at a given point of time,  $R$  is the pitting factor,  $t_p$  is the propagation time and  $i_{corr}(t)$  is the corrosion current density. For pitting corrosion,  $R$  is sampled from a Gumbel distribution according to the indications reported in [46]. The time-dependent value of  $i_{corr}(t)$  is modeled as in [47].

The pitting corrosion model proposed in [48] is used to describe the time-dependent section loss of reinforcement. Assuming hemispherical pit morphology [49][50], the residual area due to localized corrosion  $A_{R,P}$  and the pit width  $a(t)$  can be estimated. The residual reinforcement area at a given location  $A_C$  can be estimated as per Equation (8.18) [48]:

$$(8.18) \quad A_C = \left[ 1 - \frac{a(t)}{2D_0} \right] \left\{ \frac{\pi [D_0 - 2 \int_{t_i}^t 0.0116 \cdot i_{corr}(t_p) dt_p]^2}{4} - A_0 \right\} + A_{R,P}(t)$$

where  $A_0$  is the uncorroded reinforcement area.

### 8.3.3 Material Deterioration Models

Steel and concrete mechanical properties have been computed consistently with the corrosion level through specific deterioration models widely used in literature and extensively used in the previous chapters of this dissertation.

The models proposed by Du et al. [51][52] are used to compute the time dependent mechanical properties of steel, depending on the section loss  $\varphi$ . The model proposed by Andisheh et al. [53], based on the model of Mander et al. [54] is used to include corrosion effects on confined concrete, updating the compressive strength  $f_c^c$  and the ultimate strain  $\varepsilon_{ul}^c$  of confined concrete depending on the deterioration level.

#### 8.4 EXAMPLE APPLICATIONS AND VALIDATION

In order to explore the capabilities of the Life Cycle DDBA approach discussed in the previous sections, two structural configurations are initially considered for the assessment in the transverse direction, using both the DDBA method and refined NLTHAs. Previous research [16] highlighted that the DDBA method is less efficient for bridges with significant interaction between piers and bearing devices. However, given that this condition is common in existing Italian bridges, both the case studies herein considered fit into this category. In order to provide additional insights, one of the case studies has been designed according to modern durability and design standards, while the other one is inspired by an existing configuration from the 60s.

Finally, to investigate if different severity of the exposure conditions affects the validity of the approach, three different exposure conditions have been considered for the existing bridge configuration. Note that the results are scrutinized considering the  $D/C$  ratios identified using DDBA and NLTHAs approaches, and the fragility parameters obtained with both. The analysis of  $D/C$  ratios over selected stripes allows to identify the capabilities of the simplified approach to capture the inelastic response of the bridge, while the fragility parameters obtained for time-dependent fragility curves represent the starting point of seismic risk and loss assessment and discrepancies will propagate in the latter.

##### 8.4.1 Case Studies

Two different case studies, named Bridge 1 and Bridge 2, have been considered to test the capabilities of the Life Cycle DDBA approach. Bridge 1 has been extensively analyzed in Chapters 5 and 6, to which the reader may refer for a detailed description of structure and environmental exposure. Two 30 m spans are present. The deck is composed of six PC beams resting on elastomeric bearing devices, with a 0.25 m thick upper slab. The bent is composed of three columns with square sections with a connecting cap beam. The columns have 0.9 x 0.9 plan dimensions and are 8 m high. 36  $\varphi 24$  bars have been used as longitudinal reinforcement of the sections, while four legs  $\varphi 10$  stirrups with spacing 150 mm have been used as transverse reinforcement. The nominal concrete cover  $c_{nom}$  to longitudinal reinforcement was 60 mm, according to the Eurocode prescriptions. A  $w/c$  ratio of 0.45 has been assumed.

Bridge 2 corresponds to the C4 configuration analyzed in Chapter 7. The structure includes four spans, each one of them around 30 m long. Each of the three piers is 13.31 m high. The deck is composed of four PC beams resting on unbolted neoprene pad bearings, with an upper slab. The configuration of the pier is one of the main challenging features of the bridge, presenting two 3300x700 mm walls connected by a 0.6 m thick slab on their head. The plastic hinge zone of the walls is reinforced with a 0.36% of longitudinal bars,  $\varphi 8$  stirrups with spacing 200 mm have been used as transverse reinforcement.  $c_{nom}$  is equal to 43 mm. A w/c ratio of 0.5 has been assumed. A schematic representation of the two case studies is proposed in Figure 8.4. In order to obtain the representative response of an existing bridge in aggressive environments, the Life Cycle performance of Bridge 2 has been studied for conditions equivalent to XS1, XS2 and XS3 Eurocode Exposure Classes. XS1 corresponds to environmental exposure to airborne salts, without direct contact with sea water. XS2 represents structures with submerged parts. XS3 corresponds to splash and spray zones. The RVs distributions considered to model each exposure scenario can be found in Table 5.1 (Bridge 1) and Table 7.5-Table 7.7 (Bridge 2).

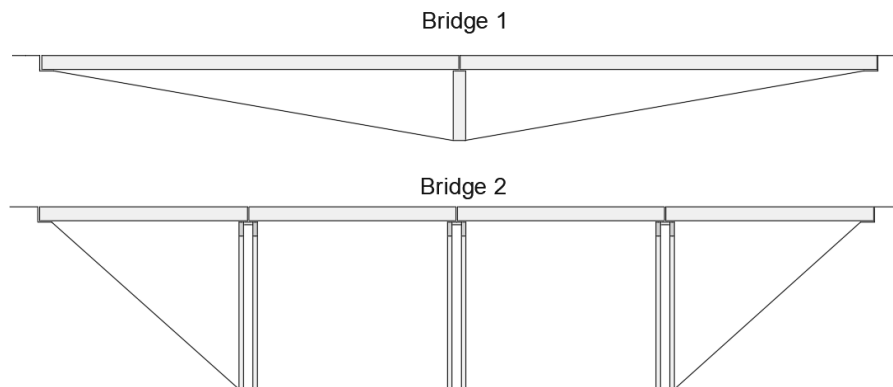


Figure 8.4. Structural configuration of the case studies considered for validation.

#### 8.4.2 Numerical Modeling for NLTHAs

The numerical models for NLTHAs have been developed using the OpenSees library [55]. According to the current practice, the superstructure is modeled using elastic beam elements. Pier columns have been modeled using the *BeamwithHinges* element that, due to their formulation which allows the inelasticity to develop over a specified plastic hinge length, allows a consistent comparison with the DDBA. The cyclic behavior of plastic hinges is represented using the *Hysteretic* material. *ZeroLength* elements have been used to describe the response of bearing devices. To provide a fair comparison with DDBA results, NLTHAs have been performed in transverse direction. The approaches for ground motion selection, damage states definition and fragility analysis are consistent with those used for the DDBA.



### 8.4.3 Bridge 1 – D/C ratios

A first comparison between DDBA and NLTHAs is carried out by computing the discrepancy between  $D/C$  ratios predicted by the two approaches. The discrepancy between the results is quantified computing a  $\beta$  coefficient defined as:

$$(8.19) \quad \beta = \frac{D/C_{DDBA}}{D/C_{NLTHA}}$$

where  $D/C_{DDBA}$  and  $D/C_{NLTHA}$  are the  $D/C$  ratios predicted for a given ground motion by DDBA and NLTHAs respectively. Note that only the first 8 IM levels have been considered due to the collapse condition identified for IM levels 9 and 10 using the DDBA approach (not associated with a specific  $D/C$  value).

Prior to comparing the  $D/C$  values obtained using the two approaches, the evolution of the DBPP curve has been evaluated by comparing  $S_d$  and  $S_a$  values corresponding to the ultimate point of the curve over time. The comparison has been reported in Figure 8.5. The results highlight that deterioration effects are negligible for  $t < 40$  years and become evident after that time.  $S_d$  and  $S_a$  values over time have been normalized to their initial value, revealing that corrosion effects are more important for  $S_d$  than for  $S_a$ , which can be intended as proxies for ductility and strength, with reduction up to around 20% and 15% respectively.

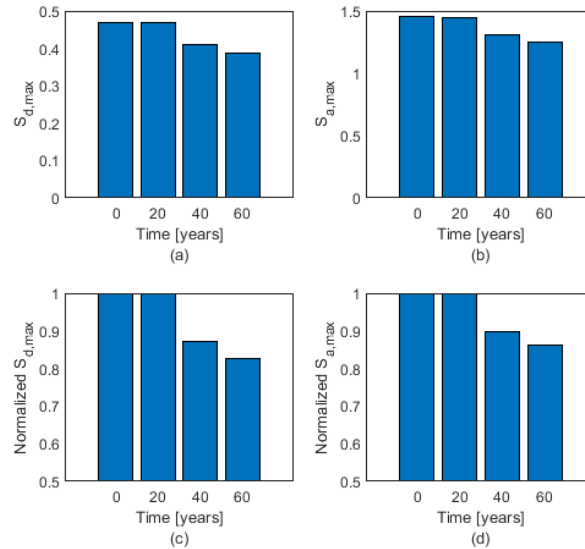


Figure 8.5. (a-b) Maximum  $S_d$  and  $S_a$  values of Bridge 1 DBPP curve over time, (c-d) maximum  $S_d$  and  $S_a$  values over time normalized on the initial value. XS3 class.

The  $\beta$  values obtained for the four DSs, for four different times along the life of the structure are represented in Figure 8.6-Figure 8.9. The results reveal that, over the different stripes, the median response obtained through the DDBA approach is aligned with the NLTHA results, with  $\beta$  values close to 1. It can be observed that the DDBA approach tends to underestimate the demand on the structure up to 40% for the lowest IM levels, while good agreement is found for the other IM levels. It is also worth noting that the seismic response estimated for lower IM levels tends to be less dispersed than for higher IM levels. The level of dispersion over the range of IM levels tends to be similar for each DS. Finally, it has been observed that these trends are confirmed also including deterioration phenomena in the analysis.

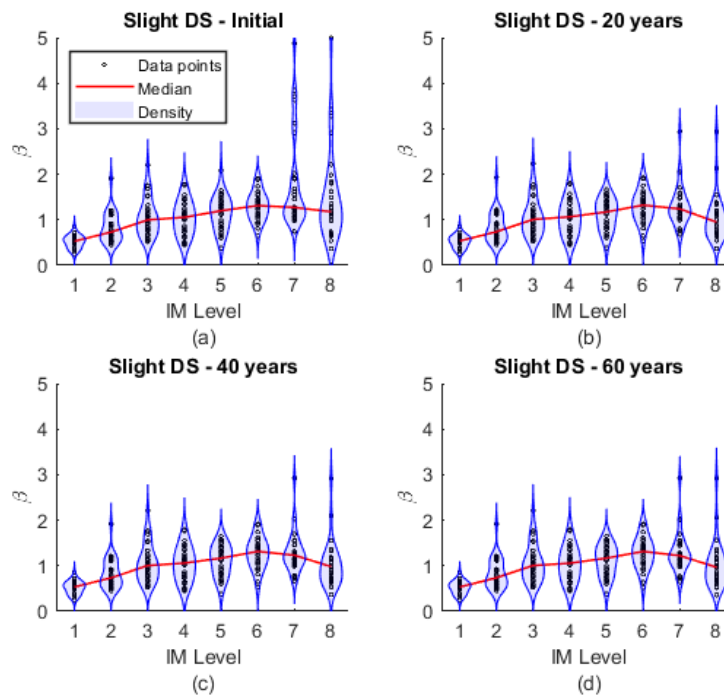


Figure 8.6.  $\beta$  ratios for Slight Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 1.

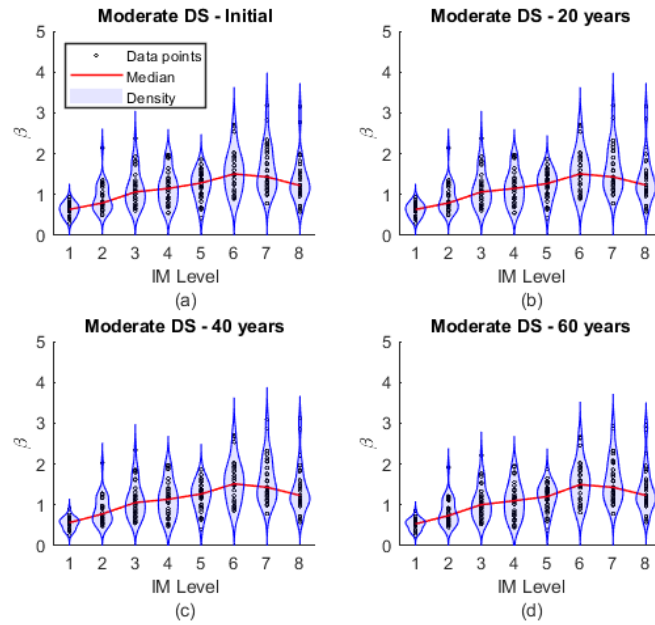


Figure 8.7.  $\beta$  ratios for Moderate Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 1.

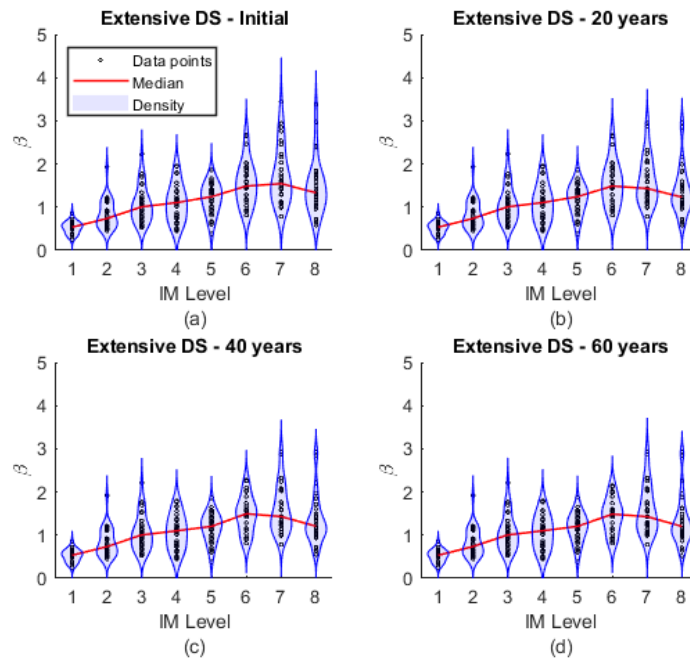


Figure 8.8.  $\beta$  ratios for Extensive Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 1.

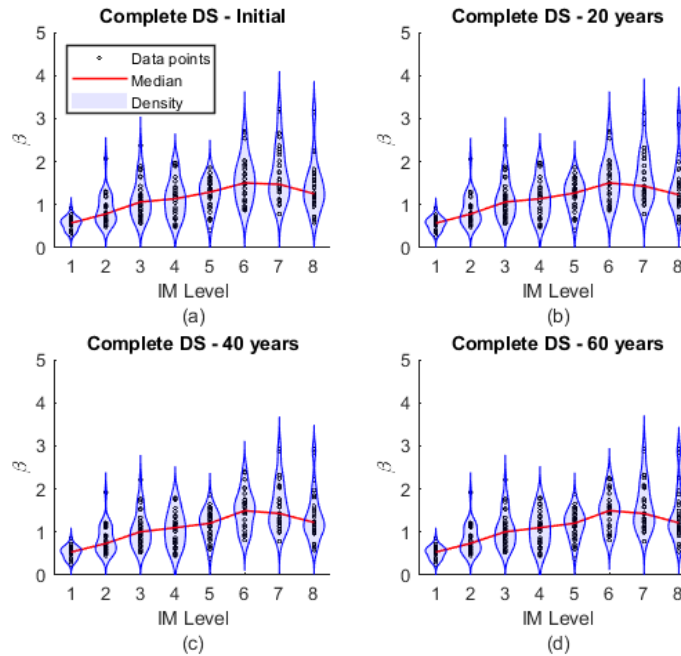


Figure 8.9.  $\beta$  ratios for Complete Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 1.

#### 8.4.4 Bridge 2 – D/C ratios

The results obtained for Bridge 2 have been scrutinized following the same approach applied to those of Bridge 1. For the sake of brevity, only the results obtained for the XS3 Exposure Class will be discussed in this section, while those related to XS1 and XS2 Exposure Classes can be found in Appendix D. The evolution of the DBPP curve has been reported in Figure 8.10. The results highlight that deterioration effects are much more influential than for Bridge 1, as indicated by the significant reduction of  $S_d$  and  $S_a$  over time. The comparison is clear if normalized quantities are taken into consideration, highlighting how the reduction of  $S_d$  and  $S_a$  values is up to around 60% and 45% respectively. It is also worth noting that the time-variant DBPP curve confirms that corrosion tends to affect more the ductility of corroded RC members than the strength.

The  $\beta$  values obtained for Bridge 2 are represented in Figure 8.11-Figure 8.14. It is worth pointing out that DDBA tends to systematically underestimate Slight DS seismic response for Bridge 2 in XS3 exposure condition, with differences up to 50%. For the other three DS, it can be observed that the demand estimation obtained using the DDBA approach is well aligned with the NLTHAs up to 20 years, with dispersion level similar to that of Bridge 1, while for  $t=40$  years and  $t=60$  years the simplified approach tends to underestimate the demand on the structure, with differences up to 50%.

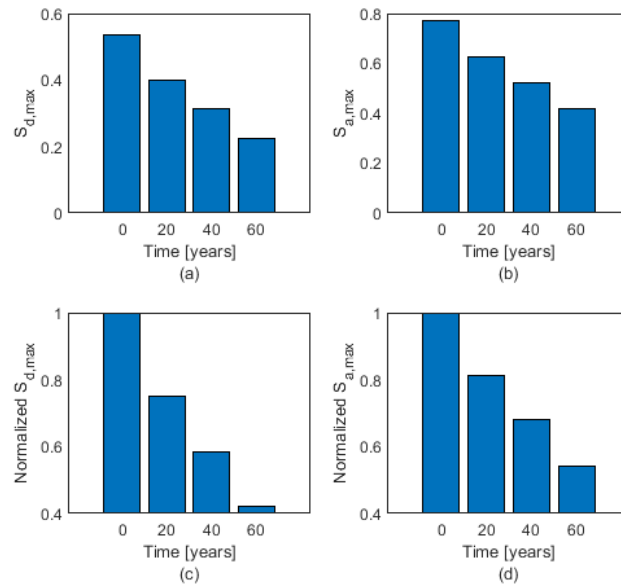


Figure 8.10. (a-b) Maximum  $S_d$  and  $S_a$  values of Bridge 2 DBPP curve over time, (c-d) maximum  $S_d$  and  $S_a$  values over time normalized on the initial value. XS3 Exposure class.

The reason for this discrepancy, which was expected according to previous findings on DDBA applications to this type of configuration [16], is the inherent limitation of the method for describing the response of bridges with significant interaction between piers and bearings. In the specific case, the walls present in the piers are extremely stiff in transverse direction and their interaction with unbolted neoprene pad bearings (representing a deformable layer) is complex to describe through simplified relationships. Unbolted neoprene pad bearings are, in fact, characterized by linear viscoelastic behavior followed by a frictional behavior [16]. Elastic stiffness  $K_{el}$  and friction force  $F_f$  used to characterize the mechanical behavior of unbolted neoprene pad bearings according to [16] are reported in Table 8.3. Bridge 2 is therefore characterized by a pier configuration complicated to describe through a simplified approach and the progression of deterioration increases the difficulty of the task. The combination of these issues motivates the discrepancy that has been observed in the seismic response estimation through DDBA. The analysis of Bridge 2 represents a valid stress test for the DDBA, highlighting the challenges in the application of the method and providing a guide for future research.

Table 8.3. Characteristics of the unbolted neoprene pads of Bridge 2.

| Elastic Stiffness $K_{el}$ [kN/m] | Friction Force $F_f$ [kN] |
|-----------------------------------|---------------------------|
| 5272                              | 271                       |

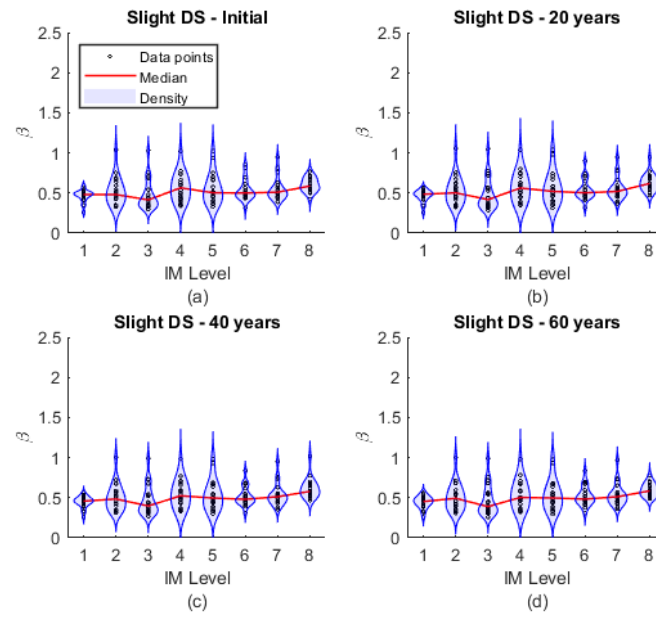


Figure 8.11.  $\beta$  ratios for Slight Damage State, for  $t=0, 20, 40, 60$  years. XS3 exposure class. Bridge 2.

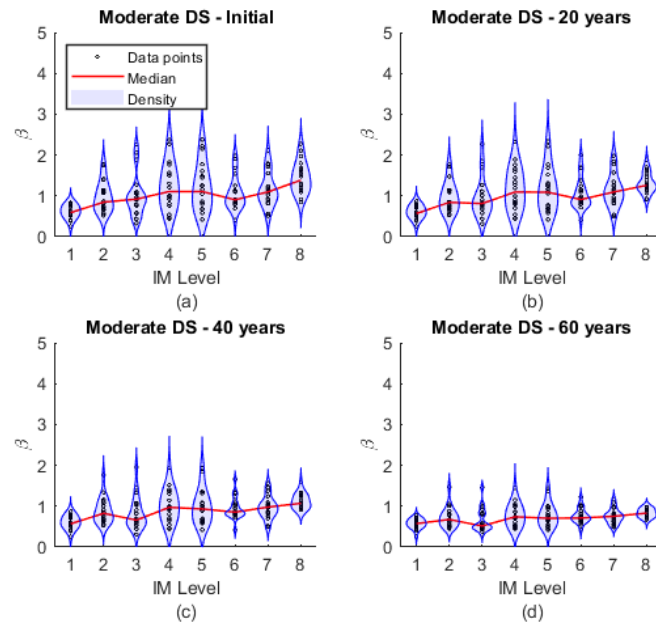


Figure 8.12.  $\beta$  ratios for Moderate Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 2.

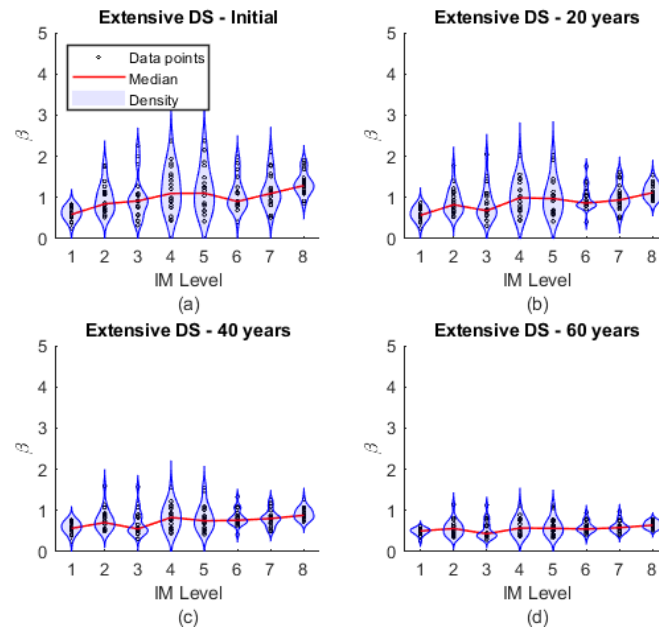


Figure 8.13.  $\beta$  ratios for Extensive Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 2.

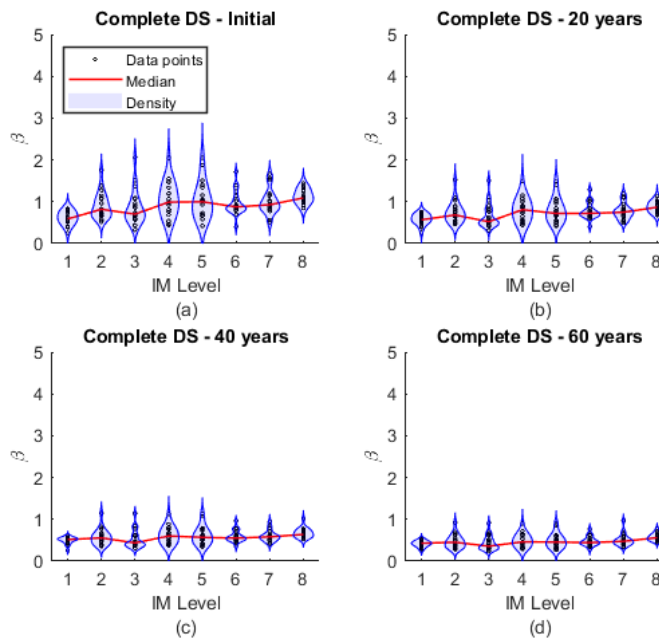


Figure 8.14.  $\beta$  ratios for Complete Damage State, for  $t=0, 20, 40, 60$  years. XS3 class. Bridge 2.

#### 8.4.5 Error in Fragility Parameters Estimation

To provide further insights on the accuracy of the DDBA approach, the errors in fragility estimation have been computed considering both  $\theta$  and  $\beta_{IM}$  parameters, according to Equations (8.20)-(8.21):

$$(8.20) \quad err_{\theta} = \frac{\theta_{DDBA} - \theta_{NLTHA}}{\theta_{NLTHA}} \cdot 100 [\%]$$

$$(8.21) \quad err_{\beta} = \frac{\beta_{DDBA} - \beta_{NLTHA}}{\beta_{NLTHA}} \cdot 100 [\%]$$

where  $\theta_{DDBA}$  is the median fragility estimated using the DDBA,  $\theta_{NLTHA}$  is the median fragility estimated through NLTHAs,  $\beta_{DDBA}$  is the dispersion estimated with the DDBA approach and  $\beta_{NLTHA}$  is the dispersion estimated using the NLTHAs. Table 8.4 reports the error values for the two case studies, for the exposure conditions considered for the Life Cycle assessment, for Moderate, Extensive and Complete DS. A few conclusions on the accuracy of the approach can be drawn based on the results. The structural configuration of Bridge 1 is significantly less complex than that of Bridge 2, with a direct influence on the estimates of the fragility parameters. An overall satisfactory agreement between NLTHAs and DDBA can be observed for Bridge 1, especially for the estimation of  $\theta$ , while the results in terms of  $\beta_{IM}$  are less satisfactory. The errors on the estimation of  $\beta_{IM}$  can be explained considering the discrepancy in the seismic demand estimation found for lower IM levels. The average error on the Life Cycle assessment is acceptable, highlighting a potential application for large scale Life Cycle assessment. At the same time, the complex configuration of the piers of Bridge 2, and the interaction between piers and unbolted neoprene pad bearings lead to an increasing error level. It is worth noting that, considering the complexity involved in the case study, the errors in the estimation of fragility parameters may still be accepted. Interesting insights can be retrieved observing the Life Cycle performance in different environmental exposure conditions. For the XS1 exposure class, the errors do not vary significantly over time, highlighting a stable performance of the structure due to the relatively low level of corrosion. For XS2 and XS3 exposure classes, more demanding in terms of Life Cycle deterioration, it is interesting to note how the DDBA approach progressively overestimates the fragility parameters, probably because of the interaction between corroded piers and bearing devices. While the average errors on Life Cycle performance are still at acceptable levels, the error level at 60 years for DS3 and DS4 is significant. The same trend was not observed in Bridge 1 due to the modern design standards applied in the definition of the prototype, limiting the corrosion progression over time.

The DDBA approach, given the significant saving in terms of computational onus compared to a complete set of NLTHAs in the MSA framework, appears to be suitable for large scale Life Cycle assessment of bridge structures and networks, even if some limitations are present.



Bridge structures that present significant complexity in the structural configuration may be better assessed through refined approaches such as NLTHAs. The prediction of the structural performance for existing bridges may not include time instants that imply high deterioration levels, due to the apparent increase of the error level in the presence of severe corrosion. Overall, the discrepancies observed in the fragility estimation could be acceptable considering the unfavorable configuration of the bridges and a large scale application oriented towards the identification of structures needing a refined assessment.

Table 8.4. Errors in the estimation of fragility parameters.

| Structure                        | Exposure Class | Time [years] | DS2<br>err <sub>0</sub> | DS2<br>err <sub>β</sub> | DS3<br>err <sub>0</sub> | DS3<br>err <sub>β</sub> | DS4<br>err <sub>0</sub> | DS4<br>err <sub>β</sub> |
|----------------------------------|----------------|--------------|-------------------------|-------------------------|-------------------------|-------------------------|-------------------------|-------------------------|
| Bridge 1                         | XS3            | 0            | -11%                    | 25%                     | -20%                    | 49%                     | -14%                    | -2%                     |
|                                  |                | 20           | -15%                    | 21%                     | -14%                    | 20%                     | -4%                     | 24%                     |
|                                  |                | 40           | -15%                    | 21%                     | -1%                     | 20%                     | 4%                      | 22%                     |
|                                  |                | 60           | -14%                    | 13%                     | -1%                     | 23%                     | 0%                      | 7%                      |
| Average errors on the life cycle |                |              | -13%                    | 20%                     | -9%                     | 28%                     | -3%                     | 13%                     |
| Bridge 2                         | XS1            | 0            | -36%                    | -37%                    | -23%                    | 14%                     | -18%                    | -2%                     |
|                                  |                | 20           | -36%                    | -37%                    | -23%                    | 14%                     | -18%                    | -2%                     |
|                                  |                | 40           | -32%                    | -31%                    | -23%                    | 14%                     | -8%                     | 7%                      |
|                                  |                | 60           | -32%                    | -31%                    | -23%                    | 14%                     | -8%                     | 7%                      |
| Average errors on the life cycle |                |              | -34%                    | -34%                    | -23%                    | 14%                     | -13%                    | 3%                      |
| Bridge 2                         | XS2            | 0            | -36%                    | -37%                    | -23%                    | 14%                     | -18%                    | -2%                     |
|                                  |                | 20           | -26%                    | -18%                    | -17%                    | 14%                     | 17%                     | 5%                      |
|                                  |                | 40           | -22%                    | -23%                    | 2%                      | 29%                     | 20%                     | 22%                     |
|                                  |                | 60           | 19%                     | -7%                     | 73%                     | 39%                     | 50%                     | 5%                      |
| Average errors on the life cycle |                |              | -16%                    | -21%                    | 9%                      | 24%                     | 17%                     | 5%                      |
| Bridge 2                         | XS3            | 0            | -36%                    | -36%                    | -23%                    | 14%                     | -18%                    | -2%                     |
|                                  |                | 20           | -22%                    | -26%                    | -7%                     | 27%                     | 35%                     | 21%                     |
|                                  |                | 40           | -11%                    | -29%                    | 13%                     | 9%                      | 29%                     | 17%                     |
|                                  |                | 60           | 19%                     | -7%                     | 73%                     | 39%                     | 50%                     | -5%                     |
| Average errors on the life cycle |                |              | -13%                    | -25%                    | 14%                     | 22%                     | 24%                     | 8%                      |

## 8.5 UNCERTAINTY STUDY

The DDBA approach presented and validated in the previous sections is applied to study the effect of deterioration and material related uncertainties on significant DDBA quantities, fragility curves and Seismic Risk and Loss metrics. The study is limited to the transverse response of Bridge 1 due to the higher level of accuracy observed in the validation phase and to allow a more in-depth discussion of the results.

### 8.5.1 Uncertainty Modeling

To perform the uncertainty study,  $f_c$ ,  $f_y$  and the section loss of longitudinal and transverse reinforcement at  $t=20$  years,  $t=40$  years and  $t=60$  years have been characterized as lognormally distributed RVs. Latin Hypercube Sampling (LHS) [56] has been used to generate 1000 samples of the parameters. LHS is more efficient than a Monte Carlo approach because structured stratification is imposed over the input space. The statistical properties of the section loss distributions have been retrieved from Monte Carlo simulations of the corrosion process.

### 8.5.2 Yielding and Ultimate Displacement

The first investigation carried out in the uncertainty study interested the values of  $\Delta_y$  and  $\Delta_u$ , computed through the DDBA principles using Equations (8.1)-(8.2). Interesting insights can be obtained observing Figure 8.15 (a)-(b). The value of  $\Delta_y$  appears to be almost constant over time, not affecting significantly the first branch of the member response. The result can be easily explained by the simplified expression of  $\varphi_y$  [14]:

$$(8.22) \quad \varphi_y = C \frac{\varepsilon_y}{D}$$

where  $C$  is a constant depending on the section type,  $D$  is the main dimension of the cross section and  $\varepsilon_y$  is the yielding strain of steel.  $\varepsilon_y$  is only slightly affected by corrosion due to the limited reduction of  $f_y$  in [51]. Consequently, given that  $\varphi_y$  is used to compute  $\Delta_y$  as per Equation (8.1) no significant variations of the latter are expected. This is confirmed in terms of variation of the median value normalized on its initial value, as per Figure 8.15 (d), where it can be observed how the value is almost constant over time.

Conversely, the value of  $\Delta_u$  is significantly affected by corrosion, with a reduction of the median value of around 25% passing from the initial condition to  $t=60$  years. Several factors contribute to this reduction, namely the loss of confinement due to the corrosion of the transverse reinforcement and the deterioration of longitudinal steel properties. It is interesting to note how besides the expected reduction of the median value of  $\Delta_u$ , the uncertainty related to the estimation of the value tends to increase over time as a direct consequence of increasing corrosion levels.

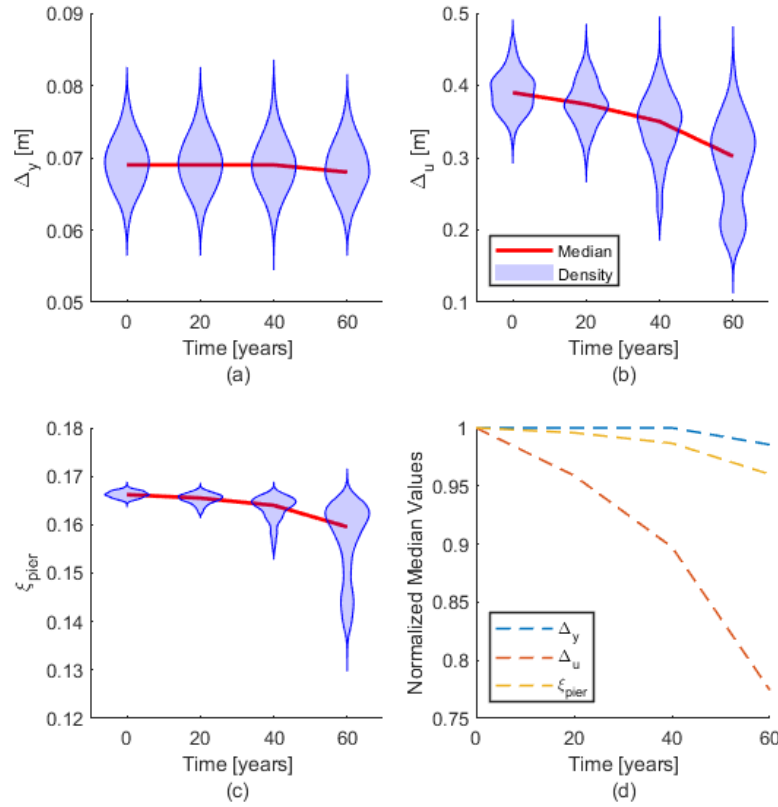


Figure 8.15. (a-c) Distribution and median values of  $\Delta_y$ ,  $\Delta_u$  and  $\xi_{pier}$  over time, (d) normalized median values of  $\Delta_y$ ,  $\Delta_u$  and  $\xi_{pier}$  over time.

This first investigation highlighted how  $\Delta_y$  and  $\Delta_u$  are affected by material and corrosion related uncertainty. While the former quantity is relatively constant over time, the latter is affected by increasing uncertainty due to a more pronounced dependence on deterioration.

### 8.5.3 Equivalent Viscous Damping

The second DDBA-related investigation is carried out by scrutinizing the values of  $\xi_{pier}$ , computed according to Equation (8.11) for the ultimate ductility of the pier. The results have been reported in Figure 8.15 (c). The reduction of the median value of  $\xi_{pier}$  is not particularly pronounced, but increasing uncertainty has been observed with the progression of simulation time. This is expected and directly related to the fact that the ultimate ductility level of the pier has been considered to compute the quantity.

Remarkably, the ductility-based damping model used in the DDBA was not developed taking into account corroded elements, so further refinement of the present approach should include alternative formulations that directly address this condition. Figure 8.15 (d) reports the median  $\xi_{pier}$  values over time normalized on the initial value, confirming that the median values are only slightly affected by corrosion effects.

#### 8.5.4 Fragility Curves

The results of the time-dependent fragility analysis are scrutinized considering the mean fragility curve and the curves corresponding to  $\text{mean} \pm \sigma$ , where  $\sigma$  indicates the standard deviation.

Considering the results obtained in the previous sections, the Slight Damage fragility curve is expected to be relatively constant over time, with relatively low uncertainty. At the same time, increasing uncertainty is expected for more severe DS and for increasing times of simulation. The time dependent fragility curves for the four DS have been reported in Figure 8.16-Figure 8.20.

Considering the same instant of the simulation, it is clear that passing from Slight DS to Complete DS the uncertainty increases, as found by previous research [28]. While for the initial condition uncertainty is related only to randomness of the material properties, the progressive increase of uncertainty is due to corrosion. Besides the uncertain characteristics of fragility curves, the progressive increase of Probability of Exceedance over the same IM level is associated with the more pronounced seismic vulnerability of the bridge structure over time.

The DDBA approach allows us to obtain interesting insights on the impact of deterioration on time-dependent assessment of bridges with a significantly reduced computational burden with respect to NLTHA-based approaches.

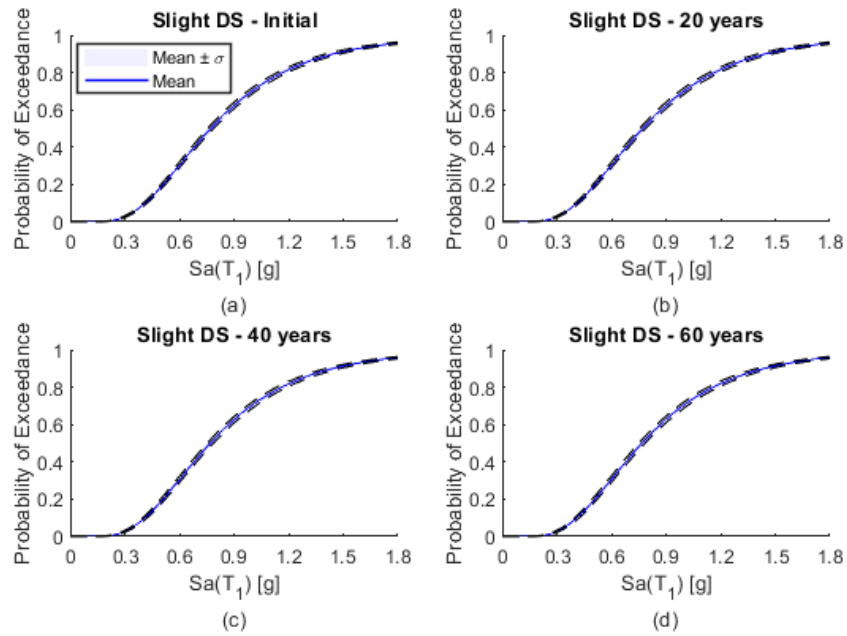


Figure 8.16. Fragility curves for Slight DS at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years.

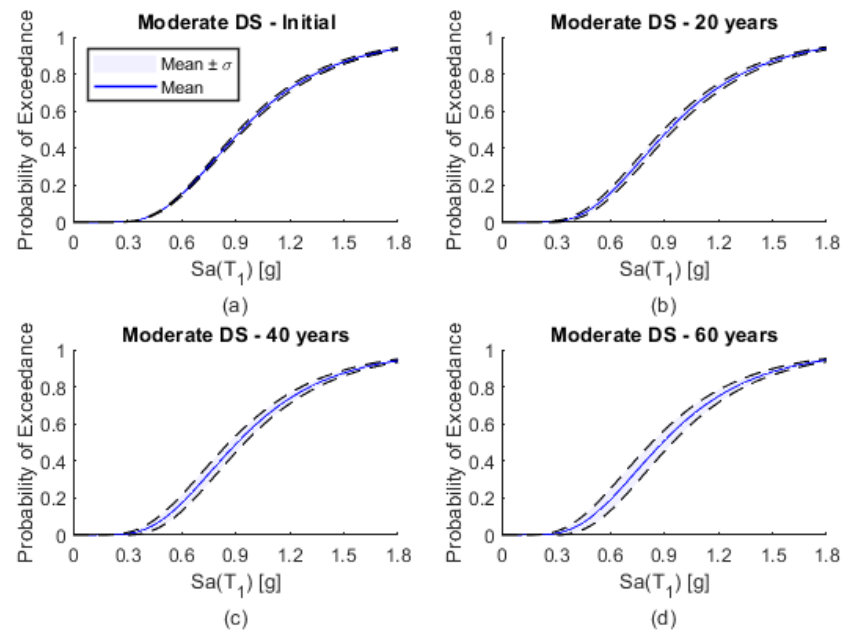


Figure 8.17. Fragility curves for Moderate DS at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years.

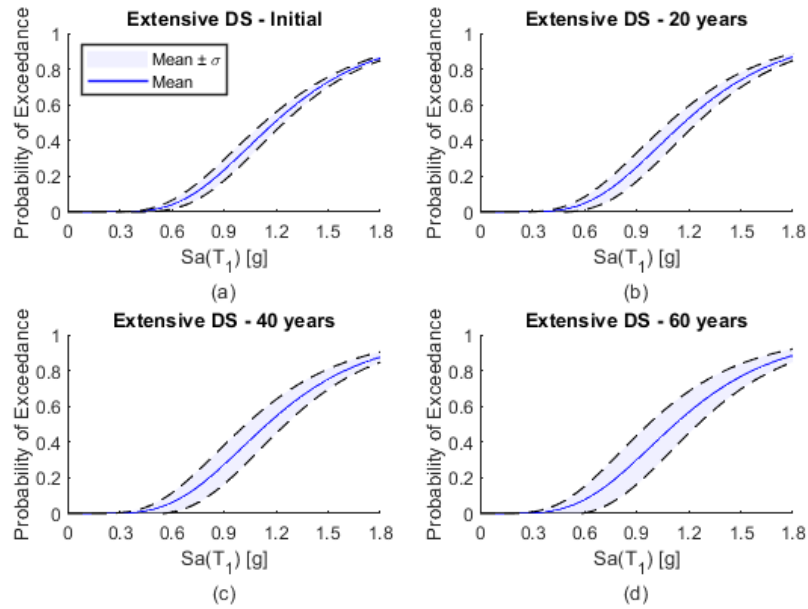


Figure 8.18. Fragility curves for Extensive DS at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years.

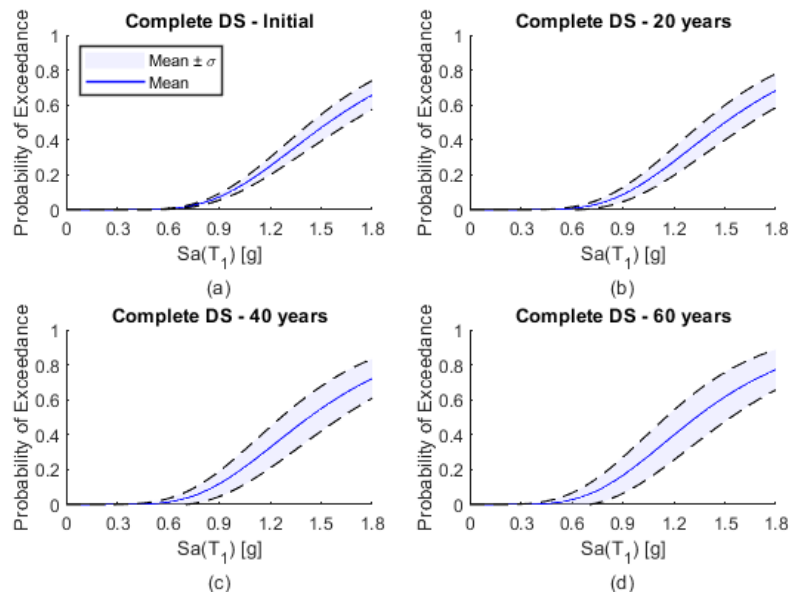


Figure 8.19. Fragility curves for Complete DS at (a) initial time, (b) 20 years, (c) 40 years and (d) 60 years.

### 8.5.5 Seismic Risk and EALR

Finally, the results are analyzed by considering the uncertainty in the time dependent values of  $\lambda_f$  and EALR. The former has been computed for each of the DS by applying Equation (8.14), while the latter is described by Equation (8.16). Both the metrics are relevant for engineers and stakeholders because they allow them to summarize the seismic risk associated with a structure at a given time of its life and to quantify the expected annual economic loss including the effects of deterioration. The results have been reported in Figure 8.20 and Figure 8.21. The  $\lambda_f$  values associated with more severe DSs are lower than those associated with less severe ones, being naturally less probable. As deterioration increases, the mean values of  $\lambda_f$  increase accordingly, following the trend observed for fragility curves, and the same applies to the uncertainty. Remarkably, Complete DS appears to be more uncertain than the other DS. Similar results have been observed in terms of EALR. The mean value of the metric increases over time, as a direct consequence of the progressive increase of seismic vulnerability due to structural deterioration, while the uncertainty increase over time is related to the progressive increase of uncertainty in corrosion modeling over time. For the sake of completeness, Table 8.5 and Table 8.6 report mean  $\lambda_f$  and EALR values over time and their normalized values. It can be observed that, in terms of Seismic Risk, Complete DS is more affected by the effects of corrosion, with an increase of mean  $\lambda_f$  over time up to 1.72 times the initial value, much higher than for the other DS. The increase of mean  $\lambda_f$  observed for Slight, Moderate and Extensive DS is respectively up to 1.01, 1.18 and 1.31 times the initial value. In terms of EALR, an increase in the mean value has been observed over time, up to 1.46 times the initial value.

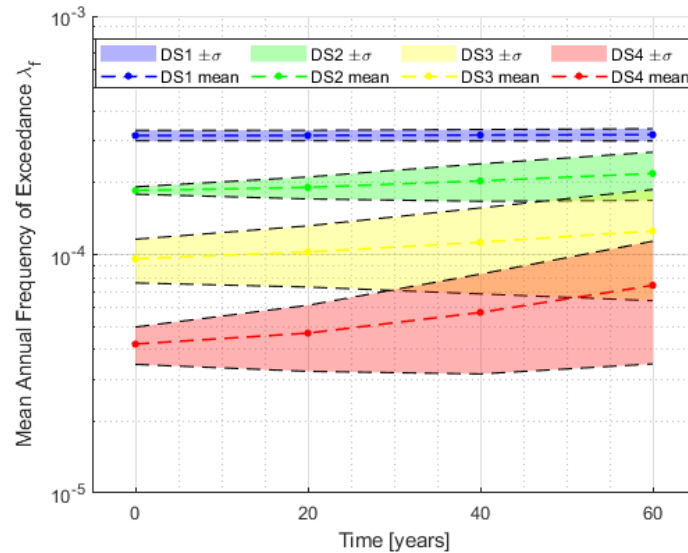


Figure 8.20.  $\lambda_f$  values over time for each DS.

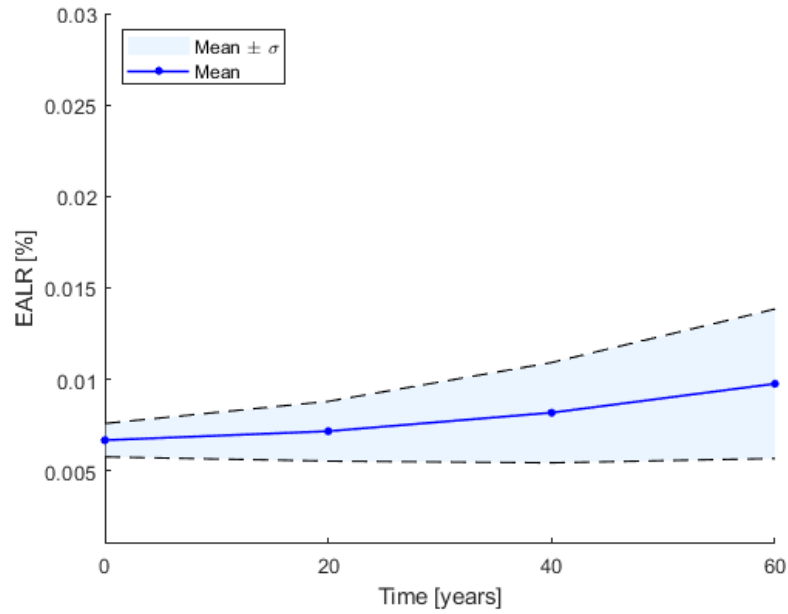


Figure 8.21. EALR values over time.

Table 8.5. Mean  $\lambda_f$  values over time for each DS. The values by branches are the  $\lambda_f$  values normalized on their initial value.

| DS  | Initial      | 20 years        | 40 years        | 60 years        |
|-----|--------------|-----------------|-----------------|-----------------|
| DS1 | 3.16E-04 (-) | 3.16E-04 (1.00) | 3.17E-04 (1.00) | 3.18E-04 (1.01) |
| DS2 | 1.86E-04 (-) | 1.91E-04 (1.03) | 2.04E-04 (1.10) | 2.19E-04 (1.18) |
| DS3 | 9.60E-05 (-) | 1.03E-04 (1.07) | 1.13E-04 (1.17) | 1.26E-04 (1.31) |
| DS4 | 4.22E-05 (-) | 4.68E-05 (1.11) | 5.72E-05 (1.36) | 7.43E-05 (1.76) |

Table 8.6. Mean EALR values [%] over time. The values by branches are the EALR values normalized on their initial value.

| Initial    | 20 years      | 40 years      | 60 years      |
|------------|---------------|---------------|---------------|
| 0.0067 (-) | 0.0072 (1.07) | 0.0082 (1.23) | 0.0098 (1.46) |

## 8.6 CONCLUSIONS

The present contribution addressed the application of DDBA principles to reduce the computational onus associated with Life Cycle assessment of bridge structures. State-of-the-art approaches for describing the structural response in a simplified yet representative



way have been collected and have been integrated into a Life Cycle assessment framework including chloride-induced corrosion and its effect on RC members. The DDBA approach has been validated through comparison with NLTHAs, revealing potential applications for large scale Life Cycle assessment. Limitations have been identified for complex structural configurations and for severe deterioration conditions, due to the interaction between corroded RC piers and unbolted neoprene pad bearings. The significant saving in the computational onus still represents an attractive feature of the approach. The DDBA approach has been applied to a fully probabilistic analysis of one bridge, in order to identify the effect of material and deterioration related uncertainties on the Life Cycle assessment results and on DDBA quantities. The results revealed that  $\Delta_j$  is relatively insensitive to corrosion, while  $\Delta_u$  is characterized by reduction of its median value and increasing uncertainty as corrosion increases. The evolution of  $\xi_{\text{pier}}$  in correspondence of  $\Delta_u$  has been studied in order to highlight possible limitations of the equivalent viscous damping model. While a slight decrease in the median value has been observed, corrosion effects are more pronounced on the uncertainty of the value. As expected, the equivalent viscous damping model incorporated in the approach does not represent well the dissipative capacity reduction typical of corroded RC members. The statistical distributions of fragility curves have been scrutinized too, highlighting that corrosion and material related uncertainties are more relevant for severe DSs and that higher corrosion levels are associated with increasing uncertainty in the seismic vulnerability estimates. Finally,  $\lambda_f$  and EALR have been analyzed. While the results obtained for  $\lambda_f$  values are conceptually similar to those obtained for fragility curves, EALR is characterized by increasing median values over time and by more pronounced uncertainty as deterioration progresses.

The present work represents a first attempt at extending the DDBA principles to time-dependent analysis. Only the transverse response, more complex to characterize for bridge structures, has been considered in this study but the extension to longitudinal analysis is straightforward. Future research may include the application of the framework to the longitudinal analysis of deteriorating structures and SSI effects. The approach could already yield reasonable results in a large scale assessment oriented towards the prioritization of interventions on bridges and networks including the effects of material and deterioration related uncertainties, but more studies are necessary to improve the predictive capacity of DDBA in a Life-Cycle-oriented framework.

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## 9. CONCLUSIONS AND FUTURE DEVELOPMENTS

### 9.1 SUMMARY AND CONCLUSIONS

This thesis explored different facets of lifetime seismic performance assessment, accounting for corrosion-induced deterioration. Corrosion represents the most important deterioration phenomenon in RC bridges, which are often exposed to harsh environmental conditions, and its effects on RC members include a number of different facets. As noted from recent bridge collapses, with structures showing widespread corrosion, structural performance assessment explicitly accounting for deterioration phenomena is crucial. Deteriorating RC structures represent a challenge for engineers all around the world, due to the complex and multivariate nature of the problem, requiring a multidisciplinary approach to be effectively overcome.

Due to their importance for road networks, the seismic assessment of RC bridges should not be limited to collapse prevention, focusing instead on the concept of seismic performance. In this sense, the most rational and rigorous approach is represented by the PBEE framework, where bridges are evaluated in terms of specific DVs. It is worth noting that, due to the presence of deterioration phenomena, bridge structural performance evolves over time, modifying the values of the DVs.

The dissertation aimed at providing original contributions improving different modules of PBEE-consistent lifetime seismic assessment, exploiting the inherent advantages of a rigorous and theoretically consistent framework allowing for treatment of uncertainty in each of its components. Validation against experimental data has been carried out for Chapters 3, 4 and 5, providing a foundation for extensive numerical analysis. It is worth noting that this is a crucial aspect and that future developments of the topic will need laboratory and field data.

The main conclusions obtained for each chapter are summarized in the following:

- Chapter 3 presented the seismic assessment of a case study representative of existing Italian bridges built in the second part of XX century. The case study has been located in three ideal Italian sites, namely Milan, Naples and L'Aquila, to investigate low, medium and high seismicity conditions. A detailed numerical model including state-of-the-art modeling of foundations and abutments and refined description of neoprene

pad bearings response has been developed using OpenSees. The deterioration scenario on the bent columns has been validated through comparison with an extensive experimental campaign conducted on the structure and negligible errors between numerical and experimental data have been found. MSA has been conducted considering 10 IM levels and considering site soil conditions through SRA. Three performance levels, UPD, SD and SF have been used to describe the structural response. The significant influence of neoprene pad bearing devices failures at UPD level confirmed the critical role of these components, while modeling the effects of corrosion made failures in bent column elements the dominant mechanism at SD and SF level. Fragility analysis revealed that corrosion leads to more pronounced seismic vulnerability of the bridge, as represented by the decrease of the median IM values of the curves and by the increased probability of exceedance of each PL. The effects of corrosion are also important in terms of decrease of seismic reliability of the case study, in fact the maximum increase of the  $\lambda_f$  values was equal to 27%, 15% and 47% for UPD, SD and SF levels respectively. The results confirmed the relevance of deterioration on seismic performance assessment of bridge structures, showing effects on dominant failure mechanisms, seismic vulnerability and seismic risk and potentially providing rational tools to aid the bridge management process.

- In Chapter 4, a refined corrosion damage modeling approach specifically designed for the seismic analysis of RC bridge piers has been presented. The fiber-based modeling strategy included inelastic buckling of rebars, low-cycle fatigue, bond-slip effects and state-of-the-art deterioration models. The approach has been validated through the simulation of corroded and uncorroded specimens subjected to quasi-static cyclic tests at University of Canterbury. The hysteretic response of the specimens has been captured by numerical models, describing realistically cyclic degradation and failures. Considering corroded specimens, the maximum errors were equal to 7%, 27% and 12% in terms of  $V_b$ ,  $k_{in}$  and  $\xi$  respectively. The effects of inelastic buckling and low-cycle fatigue have been investigated through comparison with simulations neglecting these components, revealing that their inclusion allows for better predicting the cyclic response, as revealed by the fact that the maximum error on  $\xi$  was equal to almost 30% when these phenomena were neglected. Further investigation on the performance of the modeling strategy has been carried out by computing the  $\xi$  values for different drift levels and cycles, confirming the good agreement with the experimental results. An Alternative less refined approach has been used to simulate the specimens and then compared with the Refined approach proposed in this thesis, finding that both approaches are able to correctly estimate the peak strength of the columns (even if the maximum error associated with the Alternative approach was 11%, slightly higher than the error obtained applying the Refined approach), but the latter is more effective for describing the cyclic response of corroded RC members and all the related metrics (in fact, the maximum error associated with  $\xi$  was equal to 50% using the Alternative approach and referring to corroded specimens). Additional investigation on the two



approaches involved running NLTHAs on the specimen models. The results revealed that the less refined approach underestimates the maximum drift level experienced by specimens and overestimates by 2 to 5 times the available ductility in presence of corrosion, both in the pre- and post- event phase.

- Chapter 5 investigated the mutual interaction between environmental and seismic hazard and the implications on durability and Life Cycle seismic assessment of RC bridges. A Monte Carlo simulation-based probabilistic approach has been used. Crack width is computed according to the Eurocode model. Concrete cracking is modeled through an equivalent diffusion coefficient implicitly including diffuence effects. For comparison, the results have been computed also for a benchmark condition assuming uncracked concrete. Durability issues are discussed in terms of four different CIDS and are summarized through a synthetic index. Concrete cracking significantly affects corrosion evolution in RC members, with 23% to 27% higher damage probabilities for each CIDS, highlighting how this interaction should be carefully considered for defining appropriate maintenance and management strategies of bridges in high seismicity regions. The implications on lifetime seismic performance assessment have been investigated through a PBEE-based approach, yielding fragility curves, Mean Annual Frequency of Exceedance and corresponding MRP of four DS, EALR and LCC as main results. Time variant seismic fragility curves revealed that corrosion increases the seismic vulnerability of the bridge over time in a more pronounced way if concrete cracking effects are accounted for in the simulation. Discussing this effect in terms of shift of the median fragility values, maximum reductions of 21% and 15% have been observed for DS3 respectively accounting and not accounting for concrete cracking. This result has been confirmed in terms of seismic risk assessment, where the maximum difference between the approaches has been observed for DS4 at 80 years with normalized  $\lambda_f$  values of 1.62 and 1.36 including and not including concrete cracking, respectively. In terms of loss analysis, deterioration leads to higher expected losses over time. Including concrete cracking effects in the corrosion evolutionary model leads to EALR values up to 15% higher than in the benchmark simulation. Neglecting possible concrete cracking led to slight underestimation of the LCC of the bridge. This result has been explained by considering the fact that higher discrepancies between EAL values are found after a considerable amount of time has passed, reducing the impact on LCC estimation. The exercise revealed that in high seismicity regions, cracks induced by serviceability level earthquakes may have non negligible effects on durability and time dependent seismic performance of RC bridges. To the author's best knowledge, this approach was the first one addressing the topic in a systematic way, and future refinements may include cracking influence of corrosion propagation, dynamic and self-healing effects in the crack simulation module and component-level estimation of economic losses.

- Chapter 6 presented the development of a PBEE-consistent Seismic Resilience framework tailored for deteriorating bridge structures. The components of the probabilistic framework, including time variant  $Q_0$  and probabilistic characterization of the recovery process, have been discussed. An example application of the framework to a prototype bridge configuration has been provided, carrying out the Seismic Resilience assessment for five different times along the life of the structure. The analysis of Seismic Resilience Surface, Map and R values for selected IM levels shows that Seismic Resilience significantly decreases over time. More specifically R values reductions up to 0.20 have been observed over the analyzed time horizon (80 years). Further considerations have been allowed by estimating  $T_d$ , which represents a direct and physical representation of the concept of Seismic Resilience. The maximum increase of  $T_d$ , from 9 days at the initial time to 135 days at  $t=80$  years, has been observed for the IM level corresponding to  $T_R=1000$  years. Comparing Seismic Resilience curves computed with and without considering  $Q_0$  deterioration, it has been found that including this aspect is necessary in order to describe the time-dependent seismic performance of a bridge, and maximum differences up to 30% have been observed. Finally, the effects of uncertainty in the recovery process have been evaluated, finding a maximum uncertainty of  $R \pm 0.1$ , aligned with literature results. As expected, uncertainty is higher for higher IM levels, characterized by higher recovery times, and higher expected seismic losses.
- In Chapter 7, the long-term Seismic Resilience assessment of existing bridge structure configurations has been addressed through the application of the framework introduced in Chapter 6 to four bridge configurations inspired by a real case study bridge. Three progressively more severe chloride exposure scenarios have been considered for the long-term Seismic Resilience assessment. The results have been first compared in terms of fragility curves, highlighting how Scenario 1 was associated with almost negligible effects, while Scenario 2 and Scenario 3 led to significant increase of seismic vulnerability of the four configurations over time, with a maximum reduction of median IM value of 53.07%. Long-term Seismic Resilience has been first compared in terms of mean R values over time for selected IM levels. The results highlighted that the initial seismic performance of each configuration was comparable. R values were found to be progressively lower passing from Scenario 1 to Scenario 3. Deterioration effects were found to become more evident for progressively lower IM levels as time passed, but they were always negligible up to IM levels associated with  $T_R$  lower than 5000 years. The maximum reductions of R values have been 0.41 for Scenario 2 (Configuration 2,  $T_R=10000$  years) and 0.53 for Scenario 3 (Configuration 3,  $T_R=10000$  years), confirming the direct correlation between environmental exposure severity, expected deterioration and reduction of Seismic Resilience. Seismic Resilience Maps have been presented for each configuration and exposure scenario, providing further confirmation of the results observed before. Finally, the results have been compared in terms of  $T_d$  for selected IM levels. The results observed in terms of R have been

confirmed. More specifically, the maximum increase of  $T_d$  was equal to 217 days for Scenario 2 (Configuration 2,  $T_R=10000$  years) and 279 days for Scenario 3 (Configuration 3,  $T_R=10000$  years). Each configuration is associated with high time-variant Seismic Resilience for IM levels with  $T_R$  up to 2500 years, representing rare seismic events, indicating satisfactory response to seismic events and short expected downtime. Deterioration effects should not be underestimated given their effects on expected Seismic Resilience for very rare IM levels. The investigation indicates that Seismic Resilience is a concept that could be used in prioritization schemes to optimize resource allocation for infrastructure networks management, guiding retrofit interventions and emergency management planning.

- Finally, Chapter 8 addressed the application of DDBA principles to reduce the computational onus associated with lifetime assessment of bridge structures. State-of-the-art approaches for describing the structural response in a simplified way have been collected and integrated in a lifetime assessment framework including chloride-induced corrosion and its effect on RC members. The approach has been validated through comparison with NLTHAs, revealing potential applications for large-scale lifetime assessment. Limitations have been identified for complex structural configurations and for severe deterioration conditions, due to the interaction between corroded RC piers and unbolted neoprene pad bearings. The DDBA approach tended to overestimate the demand on Bridge 1 up to 40% for the lowest IM levels, while good agreement was found for the other IM levels. DDBA systematically underestimated Slight DS seismic response for Bridge 2 in XS3 exposure condition, with differences up to 50%. For the other three DS, demand estimation obtained using the DDBA approach was well aligned with the NLTHAs up to 20 years, while for  $t=40$  years and  $t=60$  years the simplified approach tended to underestimate the demand on the structure, with differences up to 50%. In terms of fragility parameters estimation, the maximum errors for Bridge 1 were 20% for median IM value, 49% for dispersion and the maximum average errors on the life cycle were 13% for median IM value and 28% for dispersion. Considering Bridge 2, analyzed in three exposure scenarios, the maximum errors were 73% for median IM value, 39% for dispersion and the maximum average errors on the life cycle were 34% for median IM value and 34% for dispersion. The DDBA approach has been then applied to a fully probabilistic analysis of one bridge, aiming at identifying the effect of epistemic uncertainties on lifetime assessment results and on DDBA quantities.  $\Delta I_y$  was found to be relatively insensitive to corrosion-induced deterioration, while  $\Delta I_u$  was characterized by up to 25% median value reduction and increasing uncertainty as corrosion level increased. The equivalent viscous damping model commonly used for DDBA was found to not represent well the dissipative capacity reduction typical of corroded RC members. Fragility curves revealed that uncertainties are more relevant for more severe DSs, and that higher corrosion levels are associated with increasing uncertainty.  $\lambda_f$  and EALR have been considered as DVs. Complete DS

was more affected by the effects of corrosion, with an increase of mean  $\lambda_f$  over time up to 1.72 times the initial value. The increase of mean  $\lambda_f$  observed for Slight, Moderate and Extensive DS was respectively up to 1.01, 1.18 and 1.31 times the initial value. In terms of EALR, an increase in the mean value up to 1.46 times the initial value has been observed. EALR values were characterized by increasing median values and more pronounced uncertainty over time. To the author's best knowledge, the investigation represented the first attempt at integrating DDBA principles into lifetime seismic assessment. The results revealed that the approach is suitable for assessment of common bridge configurations and may be applied for first-level screening of bridge portfolios.

## 9.2 FUTURE DEVELOPMENTS

Building on the research discussed in this dissertation, a number of future research developments have been identified. As previously stated, the topic is complex and still requires significant research effort from the engineering community in order to further improve the capabilities of available tools.

The implementation of experimental data in the assessment of deteriorating structures will be further explored through application to naturally corroded PC bridge beams, representing a valuable benchmark for assessing the reliability of corrosion evolutionary models and modeling approaches and their effects on the structural performance of bridge decks. The modeling strategy developed and presented in Chapter 4 will be extended to study shear-critical piers, and further experimental validation will be carried out considering this condition. It is worth noting that this extension will be crucial in order to numerically simulate corrosion-induced failure mode shift through mechanical models, aiming at maintaining a reasonable computational effort. The Seismic Resilience Framework presented in Chapter 6 and applied to existing bridge configurations in Chapter 7 will be applied to additional structural configurations and exposure scenarios, identifying the long-term Seismic Resilience associated with different structural solutions and technologies, providing guidance for identifying effective ways to increase the resilience of road networks. The lifetime DDBA extension introduced in Chapter 8 will be strengthened and applied for preliminary screening of bridge networks, studying its possible application.

Finally, this dissertation investigated corrosion-induced deterioration, being the most important deterioration phenomenon in RC bridges. However, other deterioration phenomena should be included in the assessment framework in order to provide a comprehensive description of the time-dependent performance of bridge structures. One of the main additional phenomena that could be included is represented by aging effects on bearing devices, representing a critical aspect of the response of bridge structures.

## **APPENDIX A: COMPLETE RESULTS OF THE RINTCPE PROJECT**

### **INTRODUCTION**

This Appendix summarizes the complete results obtained in the framework of the project DPC-ReLUIIS 2022-24 WP3 on seismic risk and reliability of existing bridges. Results obtained for a case study affected by carbonation-induced corrosion have been extensively presented in Chapter 3 of this dissertation. The general framework presented in Chapter 3 and briefly recalled here has been applied to a second case study (referred to as Case Study II). The seismic response analysis has been first carried out considering fixed base numerical models, later extended to include Soil-Structure Interaction (SSI) effects. Three reference sites representative of high (L'Aquila), moderate (Naples) and low (Milan) seismic hazard regions have been considered. State-of-the-art numerical models have been developed in OpenSees [1]. Seismic response has been evaluated through Nonlinear Time History Analysis (NLTHA) carried out employing triplets of recorded accelerograms. Three progressively more severe Performance Levels (PL) have been considered, namely Usability Preventing Damage (UPD), Severe Damage (SD) and Structural Failure (SF). Each PL includes damage in piers, bearing devices, abutment backwalls and foundations (in models including SSI). Finally, seismic reliability is assessed by computing the failure rate  $\lambda_f$  associated with each PL.

### **DESCRIPTION OF CASE STUDY II**

Case Study II (corresponding to Bridge 7 in Table 1.1 and Table 1.2) is representative of a typical structural configuration found in existing Italian bridge structures built in the second half of XX century. The year of construction dates back to the early 70's. The bridge consists of four simply supported spans, each 12.5 m long. The deck is composed of five Reinforced Concrete (RC) girders, completed with a 0.3 m thick upper slab. Each girder has 0.40x0.95 m section. Each span includes four transverse beams. The deck is 10.6 m wide, and the carriageway is divided into two lanes. Unbolted neoprene pad bearings connect superstructure and substructure. Each bearing device is 0.025 m thick and has 0.4x0.6 m plan section. Multicolumn bents provide support to each span. The three bents are characterized by different heights, 14.36, 14.81 and 12.01 m respectively. Each bent is composed of two RC columns and a pier cap. The columns have square sections with 1.2 m side.

The original design documentation was not available. A simulated design process has been carried out following the design practice of the time of construction. Coherently with the probable year of construction, the design code was the D.M. 30/5/1972 “Norme tecniche alle quali devono uniformarsi le strutture in conglomerato cementizio, normale e precompresso ed a struttura metallica” (“Technical standards to which structures in concrete, normal and pre-stressed, and metal structures must conform”, translated from Italian). Concerning the bridge design code, the reference was the Circ. 14/2/1962 “Norme relative ai carichi per il calcolo dei ponti stradali” (“Standard relating to loads for the calculation of road bridges”, translated from Italian). Seismic design was addressed in a simplified way, according to Law 25/11/1962 n.1684 “Provvedimenti per l’edilizia, con particolari prescrizioni per le zone sismiche” (“Construction measures, with particular requirements for seismic regions”, translated from Italian). Experimental investigations carried out on site revealed that the mean concrete compressive strength  $f_c$  is equal to 24.13 MPa and that the mean reinforcement yielding stress  $f_y$  is 379.7 MPa. Details of the reinforcement layout for the columns are given in Table A.1. Figure A.1 displays a sketch of Case Study II.

**Table A.1. Reinforcement layout for the bent columns, obtained from the simulated design process.**

|                                           |     |
|-------------------------------------------|-----|
| <b>Total number of longitudinal bars</b>  | 28  |
| <b>Diameter of longitudinal bars [mm]</b> | 26  |
| <b>Diameter of the stirrups [mm]</b>      | 8   |
| <b>Spacing of the stirrups [mm]</b>       | 250 |
| <b>Number of legs</b>                     | 2   |



**Figure A.1. Sketch of Case Study II.**

## STRUCTURAL MODELING

The structural model of Case Study II has been developed in OpenSees [1]. A grillage model has been used to describe the deck, idealizing girders and transverse beams through elastic beam elements and the RC slab through *ShellMITC4* elements [2]. The cracked properties of RC elements have been considered in the structural model [3]. Slab and elastic beams have been connected through rigid elements. Unbolted neoprene pad bearings have been modeled using *flatSliderBearing* elements [2]. This modeling choice is explained by considering that the combination of low vertical load and small thickness of the pad usually leads to slipping failure [4]. The linear viscoelastic branch of the element response is governed by the shear stiffness  $K$ , which can be computed as in Equation (A.1):

$$(A.1) \quad K = \frac{GA}{t_r}$$

where  $G$  is the rubber shear modulus assumed as equal to 1 MPa,  $A$  is the cross section area of rubber and  $t_r$  is the total thickness of the rubber layers. The second branch of the response, characterized by friction-like behavior is controlled by the neoprene-concrete friction coefficient  $\mu$ .  $\mu$  has been estimated according to the formulation proposed by Schrage [5], yielding a value of 0.45. Bent caps have been modeled using elastic beams with cracked properties [3]. Bent columns have been modeled with distributed plasticity elements with force-based formulation employing the Gauss-Radau integration method [6]. A fiber-based approach has been used to model the plastic hinge sections. Steel fibers have been described using the *Steel02* uniaxial material [7] combined with a *MinMax* material wrapper enforcing a tensile strain limit of 0.1 [3]. *Concrete01* has been used to model the concrete cover, while *Concrete04* has been used for the concrete core. Mander's model [8] has been used to compute the confined concrete properties. P-Delta effects have been included in the model. The interaction between deck and backwall has been modeled according to [9]. The effects of foundational flexibility, considered in the SSI model, have been included through the macroelements proposed in [10][11].

## STRUCTURAL RESPONSE

As discussed in Chapter 3, the structural response has been assessed through NTLHAs with tri-component ground motions. Multiple Stripe Analysis [12] has been performed considering 10 Intensity Measure (IM) levels. 20 triplets of ground motions have been used to characterize the seismic response of the case study. The ground motions have been selected for Soil Category A [13] and Site Response Analysis considering specific foundation depths and soil profiles has been performed [14]. PLs have been characterized as in Chapter 3, and the same nomenclature is maintained in this Appendix. Table A.2 lists the Engineering Demand Parameters (EDPs) and the associated thresholds for each PL. The exceedance of the PLs is defined computing for each NLTHA the demand to capacity

ratio (D/C) associated with each component. Assuming that the bridge behaves as a series system, for each analysis the maximum D/C value among the values associated with each component is taken into account to identify the structural performance.

**Table A.2.** EDPs and thresholds for the PLs, adapted from [15].

| Component         | UPD                                 |            | SD                                  |                                       | SF                                  |             |
|-------------------|-------------------------------------|------------|-------------------------------------|---------------------------------------|-------------------------------------|-------------|
|                   | EDP                                 | Limit      | EDP                                 | Limit                                 | EDP                                 | Limit       |
| Neoprene Pads     | Relative horizontal displacement    | $d_{fr}$   | Relative horizontal displacement    | $d_{60}$                              | Relative horizontal displacement    | $d_{ms}$    |
| Piers (Ductile)   | Chord Rotation                      | $\theta_y$ | Plastic Hinge Rotation              | $\theta_y + 0.5(\theta_u - \theta_y)$ | Plastic Hinge Rotation              | $\theta_u$  |
| Piers (Brittle)   | Shear Force                         | $V_u$      | Shear Force                         | $-0.2V_u$                             | Shear Force                         | $-0.5V_u$   |
| Abutment Backwall | Relative deck-backwall displacement | $Gap$      | Relative deck-backwall displacement | $Gap + F_{bw} / k_{bw}$               | Relative deck-backwall displacement | $0.1h_{bw}$ |
| Foundations       | Top piles displacement              | $d_y$      | Top piles displacement              | $d_d$                                 | Top piles displacement              | $d_u$       |

Figures A.2-A.7 display D/C ratios, number of failures (D/C>1), critical elements and cumulative failures for each PL, for each stripe, considering fixed base and SSI models for each of the sites. The number of failures tends to decrease going from UPD to SF. A direct relationship between seismic hazard and number of failures is evident. General considerations can be drawn by discussing the type of failure characterizing each PL. UPD is dominated by neoprene pad bearings failures, while backwall failures are more important for SD. For SF, bearing devices are the governing mechanism for the site of L'Aquila, while a balanced distribution of failures mechanisms has been observed for Naples. The influence of SSI is not negligible for higher IM levels of Naples and L'Aquila. Foundational failures are in some cases the governing mechanism, highlighting how this facet of seismic response of existing bridges may be important in the assessment. The number of failures does not change dramatically, but a general reduction of D/C values has been observed, even though a precise trend has not been identified.



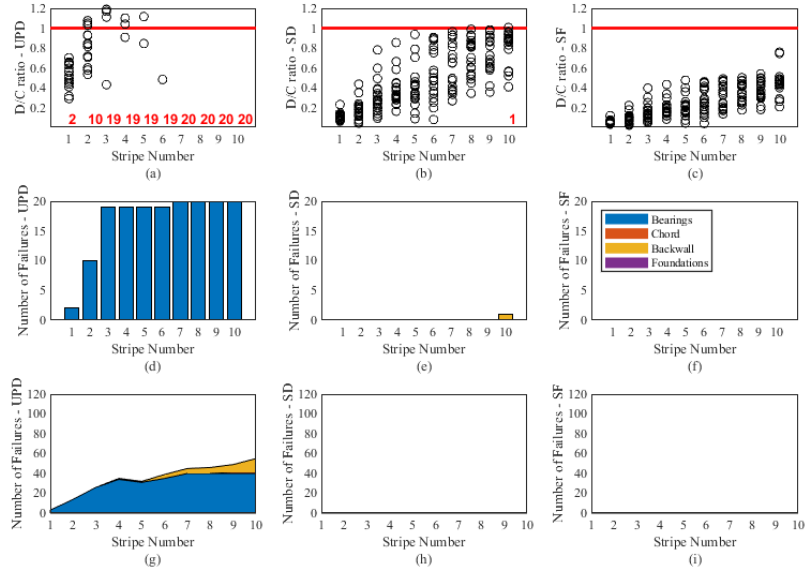


Figure A.2. Structural Response for Case Study II in fixed base condition, site of Milan. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL.

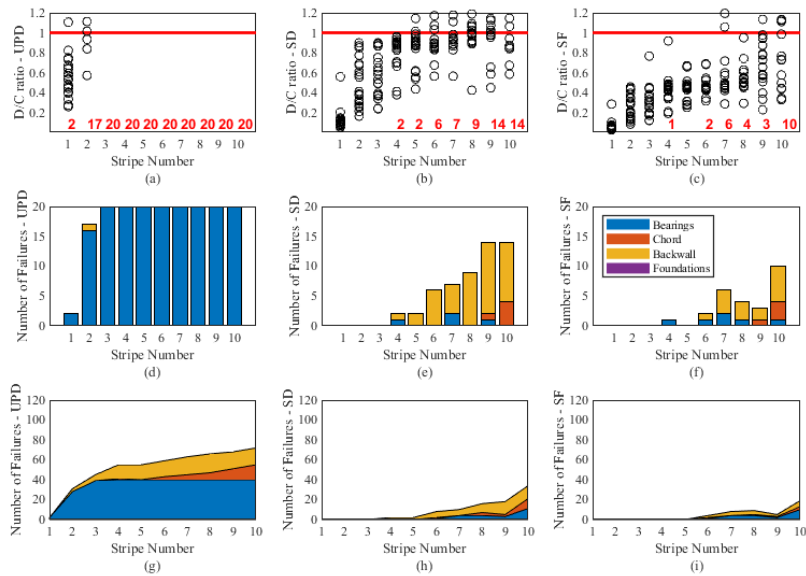


Figure A.3. Structural Response for Case Study II in fixed base condition, site of Naples. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL.

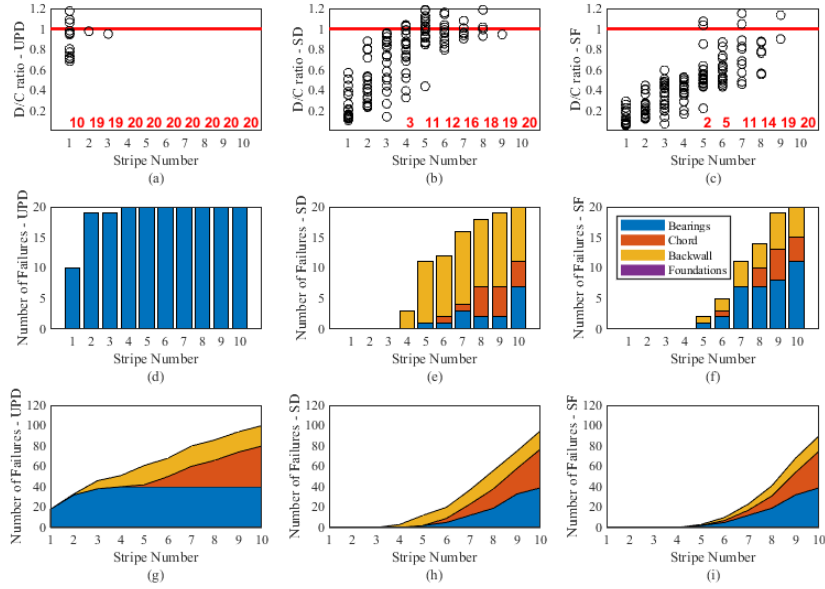


Figure A.4. Structural Response for Case Study II in fixed base condition, site of L'Aquila. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL.

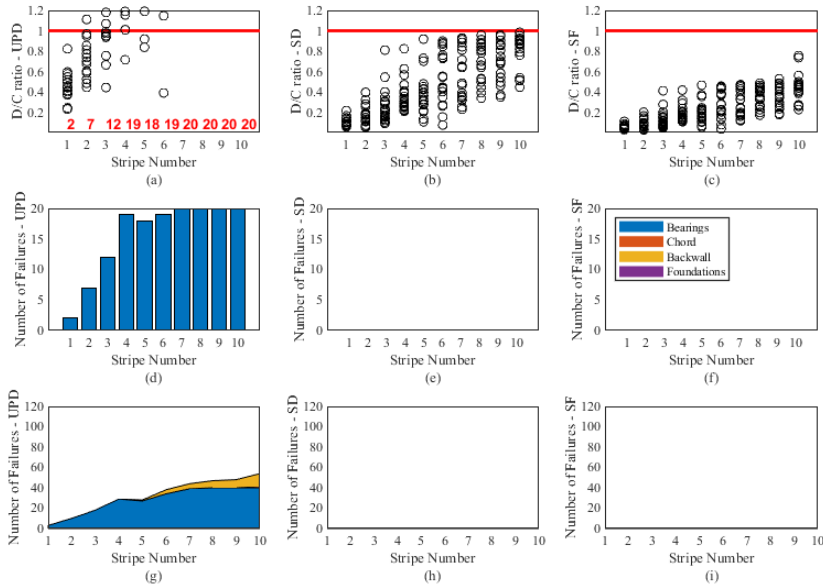


Figure A.5. Structural Response for Case Study II in SSI condition, site of Milan. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL.

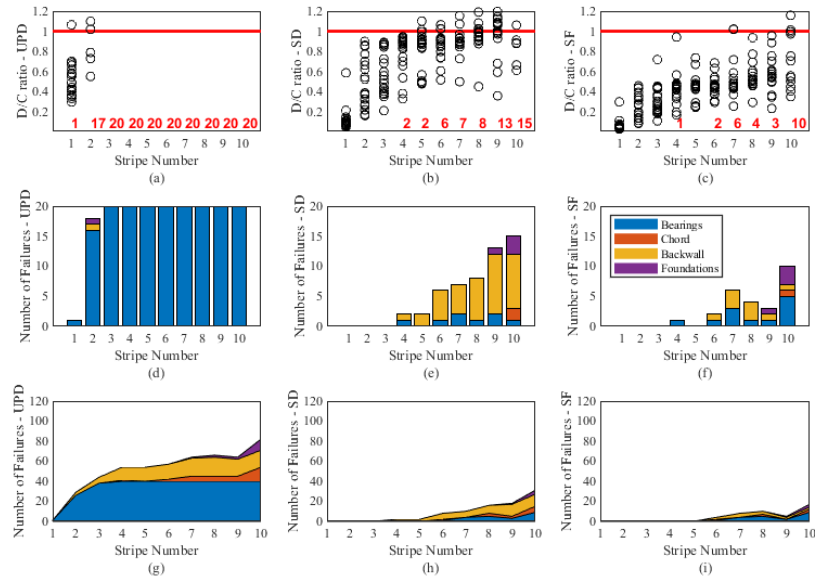


Figure A.6. Structural Response for Case Study II in SSI condition, site of Naples. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL

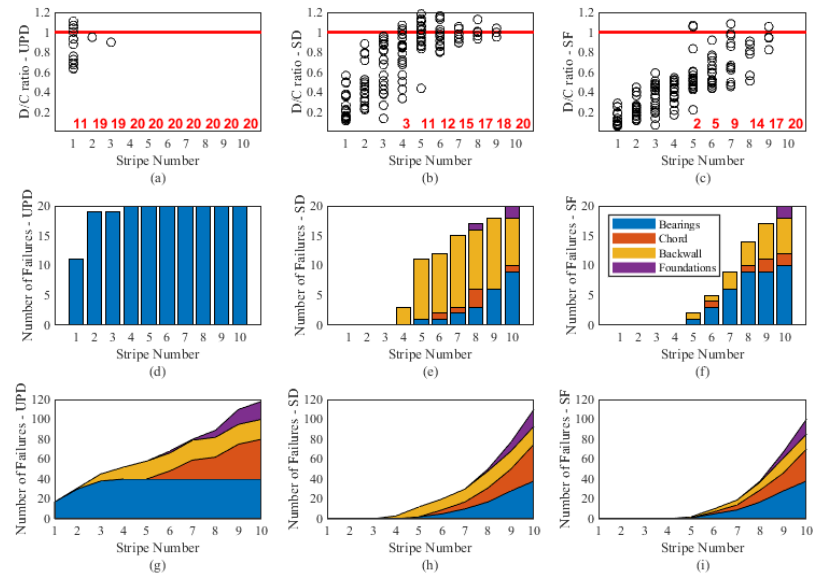


Figure A.7. Structural Response for Case Study II in SSI condition, site of L'Aquila. (a-c) D/C ratios and number of failures, (d-f) failures corresponding to maximum D/C ratio, (g-i) cumulative number of failures for each stripe and PL

### SEISMIC RELIABILITY ASSESSMENT

Seismic reliability has been evaluated computing  $\lambda_f$  for each PL as per Equation (A.2):

$$(A.2) \quad \lambda_f = \int_0^\infty P(F|IM = x) |d\lambda(IM > x)|$$

Where  $P(F|IM = x)$  is the fragility function for a given PL and  $|d\lambda(IM > x)|$  is the absolute value of the derivative of the site hazard curve. Fragility functions have been obtained through the R2R-EU software [16]. Table A.3 reports the  $\lambda_f$  values estimated for Case Study II, for fixed base and SSI models, considering each site and PL and the relative difference between SSI and fixed base models. Some considerations can be drawn from the results. The most evident consideration is that structural reliability tends to decrease as seismic hazard increases and that  $\lambda_f$  decreases passing from UPD to SF. Comparing SSI and fixed base models, it can be observed that including the SSI component in the structural models leads to a decrease in  $\lambda_f$  values in the sites of Milan and Naples. Considering the case of L'Aquila, a slight increase in the  $\lambda_f$  values associated with SD has been observed. This result can be explained by considering the vulnerability of foundations for high levels of IM. The results are complementary to those obtained for the seismic response of Case Study I. Finally, comparing the results with those obtained for modern bridges [14], it can be observed that existing bridge structures are characterized by higher seismic vulnerability, directly connected to lower seismic reliability.

**Table A.3. Failure rate values for each PL and each site in fixed base and SSI models. \*=fragility function fitting not robust enough to compute the corresponding failure rate due to the insufficient number of analyses where the structure did not exceed the performance threshold. \*\*Fragility estimated through Least-Squares Fitting method.**

| Site     | PL  | $\lambda_f$ (Fixed Base) | $\lambda_f$ (SSI) | Difference [%] |
|----------|-----|--------------------------|-------------------|----------------|
| Milan    | UPD | 6.56E-02                 | 4.50E-02          | -31.40         |
|          | SD  | 1.08E-05                 | 2.39E-03**        | -              |
|          | SF  | 6.09E-04**               | 6.35E-04**        | 4.27           |
| Naples   | UPD | 5.22E-02                 | 4.60E-02          | -11.88         |
|          | SD  | 9.87E-04                 | 9.19E-04          | -6.89          |
|          | SF  | 4.16E-04                 | 4.16E-04          | 0.00           |
| L'Aquila | UPD | *                        | 1.90E-01          | -              |
|          | SD  | 2.55E-03                 | 2.78E-03          | 9.02           |
|          | SF  | 7.16E-04                 | 6.92E-04          | -3.35          |

### CONCLUSIONS

This Appendix presented the seismic assessment of a case study representative of existing Italian bridges built during the second part of XX century. The general framework

discussed in Chapter 3 has been followed. Fixed Base and SSI detailed numerical models have been developed using OpenSees. The seismic response of the case study bridge has been studied through MSA, and the results have been thoroughly scrutinized. Seismic response has been described considering three PLs, referred to as UPD, SD and SF. UPD is dominated by neoprene pad bearings failures, while backwall failures are more important for SD. For SF, bearing devices are the governing mechanism for the site of L'Aquila, while a balanced distribution of failures mechanisms has been observed for Naples. The influence of SSI is not negligible for the higher IM levels of Naples and L'Aquila. Foundational failures are in some cases the governing mechanism. The number of failures does not change dramatically, but a general reduction of D/C values has been observed. The results reveal that SSI should be carefully addressed in the assessment of existing bridge structures in order to obtain a realistic estimation of seismic performance.

$\lambda_f$  has been used as seismic reliability metric. In terms of seismic reliability, it has been observed that  $\lambda_f$  decreases passing from UPD to SF and increases as the site seismic hazard increases. The discussion of SSI effects on  $\lambda_f$  is different depending on the site seismic hazard. For Milan and Naples, including the SSI component in the structural models leads to a decrease in  $\lambda_f$  values. Considering L'Aquila, a slight increase in the  $\lambda_f$  values associated with SD has been observed, mainly because of the vulnerability of foundations for high levels of IM. The results revealed that existing bridge structures are generally characterized by higher seismic vulnerability than modern ones, and that SSI and foundational vulnerability play a non-negligible role in high seismic hazard sites.

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## APPENDIX B: ADDITIONAL SIMULATIONS OF QUASI-STATIC CYCLIC TESTS ON RC COLUMNS

### INTRODUCTION

This Appendix presents additional simulations of quasi-static cyclic tests carried out on Reinforced Concrete (RC) columns using the modeling approach discussed in Chapter 4 of this dissertation. The simulations briefly presented and discussed in this Appendix did not include corrosion and have been carried out to validate the modeling approach when applied to RC elements in pristine condition. The characteristics of the six simulated benchmarks are summarized. The hysteretic response of each specimen is obtained by simulating the experimental testing protocol followed in laboratory. The validation of the modeling strategy in terms of global response is obtained by considering four control quantities. More insights are finally obtained by quantifying the errors in the simulation for different cycles and drift levels on two out of the six benchmark specimens.

### BENCHMARK SPECIMENS

The experimental results obtained for six benchmark specimens with different characteristics in terms of geometry, reinforcement and axial load ratio  $\nu$  have been retrieved from the PEER Structural Performance Database [1]. In order to provide meaningful validation of the modeling strategy, all of the selected benchmark specimens were characterized by flexural failure. Configuration and details of the specimens are summarized in Table B.1 and Table B.2. More details on the specimens can be found in [2]-[7]. Cantilever specimens with square, rectangular and circular sections have been considered.  $\nu$  values ranging from 0.032 to 0.113 have been considered. Observing the values of longitudinal and transverse reinforcement diameter (respectively denoted as  $\Phi_l$  and  $\Phi_t$ ), number of bars  $n_{bars}$ , spacing of transverse reinforcement  $s$  and longitudinal and transverse reinforcement percentage ( $\rho_l$  and  $\rho_t$ ) it is clear that the six benchmark specimens represent a valuable testbed for the capabilities of the modeling strategy.

Table B.1. Configuration of the benchmark specimens.

| Specimen                        | Section     | Section size [mm] | Height [mm] | Configuration | Axial Load [kN] | $\nu$ |
|---------------------------------|-------------|-------------------|-------------|---------------|-----------------|-------|
| Lehman et al. 1998, 415 [2]     | Circular    | 609.6             | 2438.4      | Cantilever    | 653.86          | 0.072 |
| Tanaka and Park 1990, No. 5 [3] | Square      | 550               | 1650        | Cantilever    | 968             | 0.1   |
| Park and Paulay 1990, No. 9 [4] | Rectangular | 400x600           | 1784        | Cantilever    | 646             | 0.1   |
| Kunnath et al. 1997, A2 [5]     | Circular    | 305               | 1372        | Cantilever    | 200             | 0.094 |
| Ohno and Nishioka 1984, L2 [6]  | Square      | 400x400           | 1600        | Cantilever    | 127             | 0.032 |
| Mo and Wang 2000, C1-1 [7]      | Square      | 400x400           | 1400        | Cantilever    | 450             | 0.113 |

Table B.2. Details of the benchmark specimens.

| Specimen                        | $\Phi_l$ [mm] | $n_{bars}$ | $\Phi_t$ [mm] | $s$ [mm] | $\varrho_l$ [-] | $\varrho_v$ [-] |
|---------------------------------|---------------|------------|---------------|----------|-----------------|-----------------|
| Lehman et al. 1998, 415 [2]     | 15.9          | 22         | 6.4           | 31.8     | 0.0149          | 0.007           |
| Tanaka and Park 1990, No. 5 [3] | 20            | 12         | 12            | 110      | 0.0125          | 0.0074          |
| Park and Paulay 1990, No. 9 [4] | 10            | 24         | 12            | 80       | 0.0188          | 0.009           |
| Kunnath et al. 1997, A2 [5]     | 9.5           | 21         | 4             | 19       | 0.0204          | 0.0092          |
| Ohno and Nishioka 1984, L2 [6]  | 19            | 8          | 9             | 100      | 0.0142          | 0.003           |
| Mo and Wang 2000, C1-1 [7]      | 19            | 12         | 6.3           | 50       | 0.0214          | 0.006           |



## NUMERICAL SIMULATION OF THE BENCHMARK SPECIMENS

The hysteretic response of each benchmark specimen has been simulated by applying the specific loading protocol for each one of them and using the numerical modeling strategy discussed in Chapter 4. Figure B.1 presents the comparison between the simulated and experimental cyclic response of each specimen. Qualitatively, the correspondence in terms of base shear  $V_b$  is good for each specimen. In terms of hysteretic response, the numerical models are in strong agreement with the experimental response.

To provide a quantitative estimation of the discrepancy between numerical simulations and experimental results for the benchmark specimens,  $V_b$ , initial stiffness  $k_{in}$  and energy dissipation  $E_d$  have been computed. To compute  $E_d$ , which is essentially the cumulative work of the simulated specimens, a trapezoidal integration rule has been used. Figure B.2 represents the values of the control parameters, computed for both simulated and experimental conditions. The relative error  $E$  for each quantity has been computed as per Equation (B.1):

$$(B.1) \quad E = \frac{Par_{numerical} - Par_{experimental}}{Par_{experimental}} \cdot 100$$

where  $Par_{numerical}$  is the value of a control parameter in the numerical simulation and  $Par_{experimental}$  represents the experimental value of the same control parameter. Table B.3 reports the  $E$  values estimated for each control parameter, for each specimen. The results confirm the strong agreement between simulation and experiment, with 21 out of 24 errors under 15% and 17 out of 24 under 10%, confirming that the modeling strategy is adequate to predict the hysteretic response of RC columns.

The performance of the modeling strategy was further investigated by computing the absolute errors for  $V_b$ ,  $K$ ,  $E_d$  and ratio between actual and initial stiffness  $K/K_{in}$  for selected drift levels (1%, 2% and 4%) for two different cycles for the Lehman et al. and for the Park and Paulay specimens. Figure B.3 and Figure B.4 represent these errors for the two specimens. The errors for the Lehman et al. specimen are always below 15% except for the first cycle at 1% drift, where slightly higher discrepancies can be noted for  $V_b$  and  $E_d$ . A similar trend can be observed for the Park and Paulay specimen. In general, errors tend to be higher for lower drift levels and to reduce for higher levels. It is worth noting that the absolute values of the control quantities for lower drift levels are relatively small if compared with the entire response, with differences that are negligible in absolute terms. This investigation confirms the effectiveness of the modeling strategy in uncorroded conditions, providing a realistic simulation of the response.

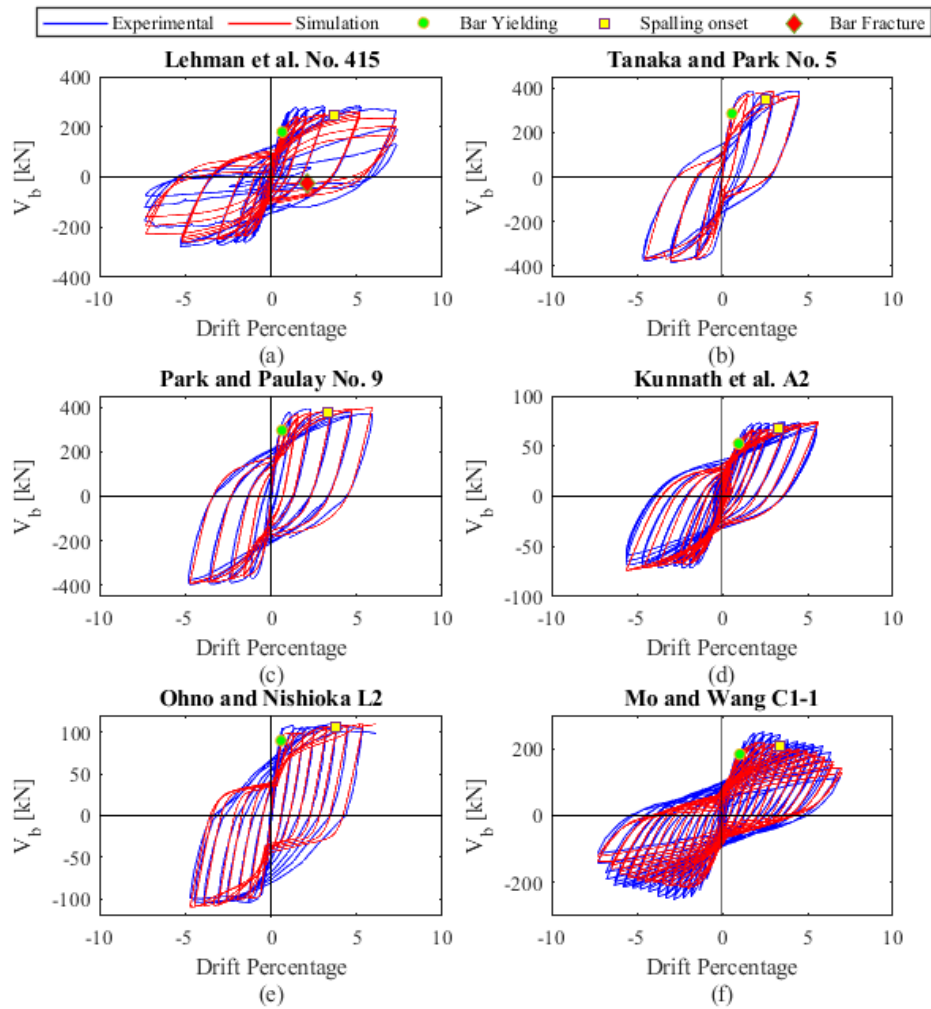


Figure B.1. Simulated vs Experimental cyclic response for (a) Lehman et al. 415, (b) Tanaka and Park No. 5, (c) Park and Paulay No. 9, (d) Kunnath et al. A2, (e) Ohno and Nishioka L2, (f) Mo and Wang C1-1 specimens.

Table B.3. Estimation of the errors between numerical simulations and experimental results.

| Specimen                        | $E_{vb+}$ [%] | $E_{vb-}$ [%] | $E_{kin}$ [%] | $E_{Ed}$ [%] |
|---------------------------------|---------------|---------------|---------------|--------------|
| Lehman et al. 1998, 415 [2]     | -8.26         | -5.66         | -20.95        | 6.25         |
| Tanaka and Park 1990, No. 5 [3] | -0.63         | -1.71         | 2.5           | -3.45        |
| Park and Paulay 1990, No. 9 [4] | -0.58         | -1.34         | -16.65        | -1.20        |
| Kunnath et al. 1997, A2 [5]     | 0.56          | 3.10          | -11.51        | -0.18        |
| Ohno and Nishioka 1984, L2 [6]  | 0.43          | 3.84          | -17.99        | -14.12       |
| Mo and Wang 2000, C1-1 [7]      | -12.14        | -13.12        | 3.46          | 3.61         |

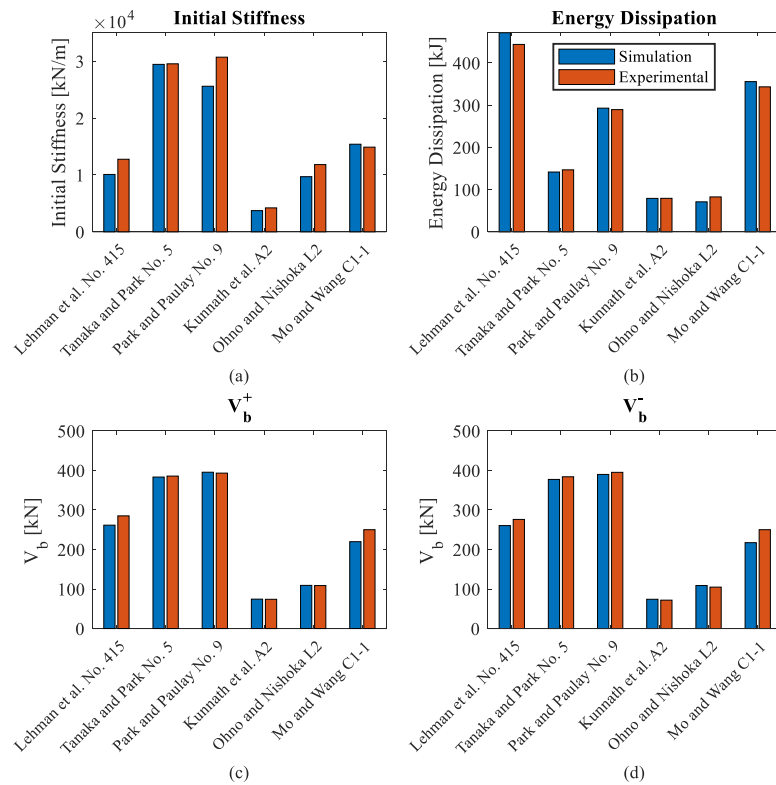


Figure B.2. Numerical vs Experimental values of the control quantities for the benchmark specimens. (a) Initial Stiffness, (b) Energy Dissipation,  $V_b$  in (c) positive and (d) negative direction.

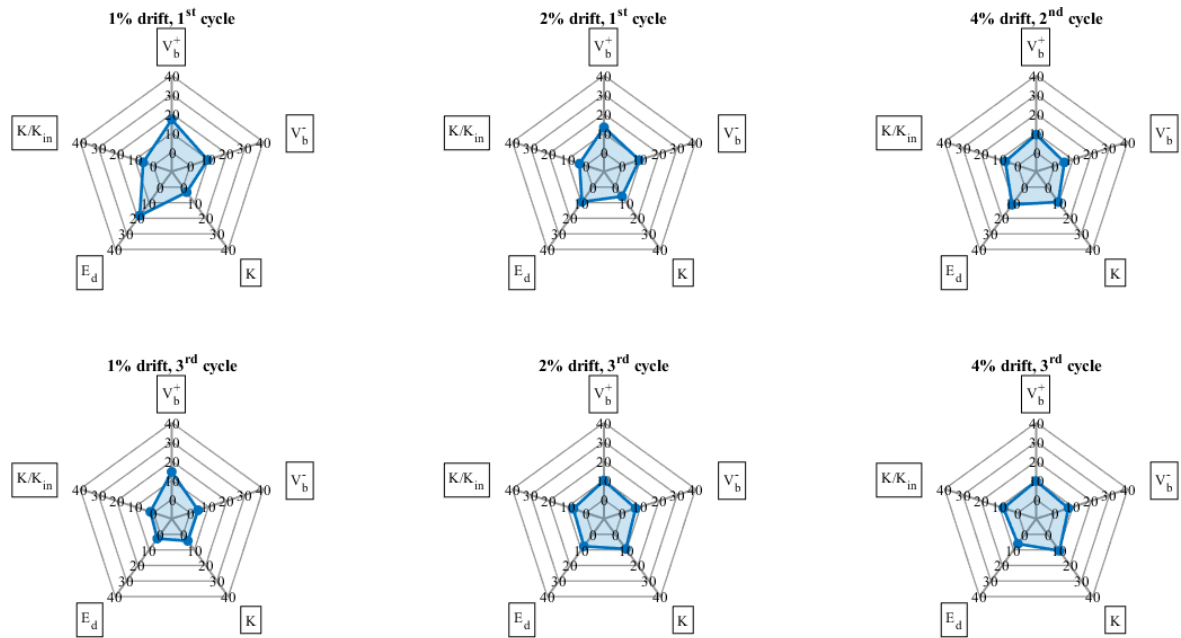


Figure B.3. Absolute errors for control quantities, Lehman et al. specimen.

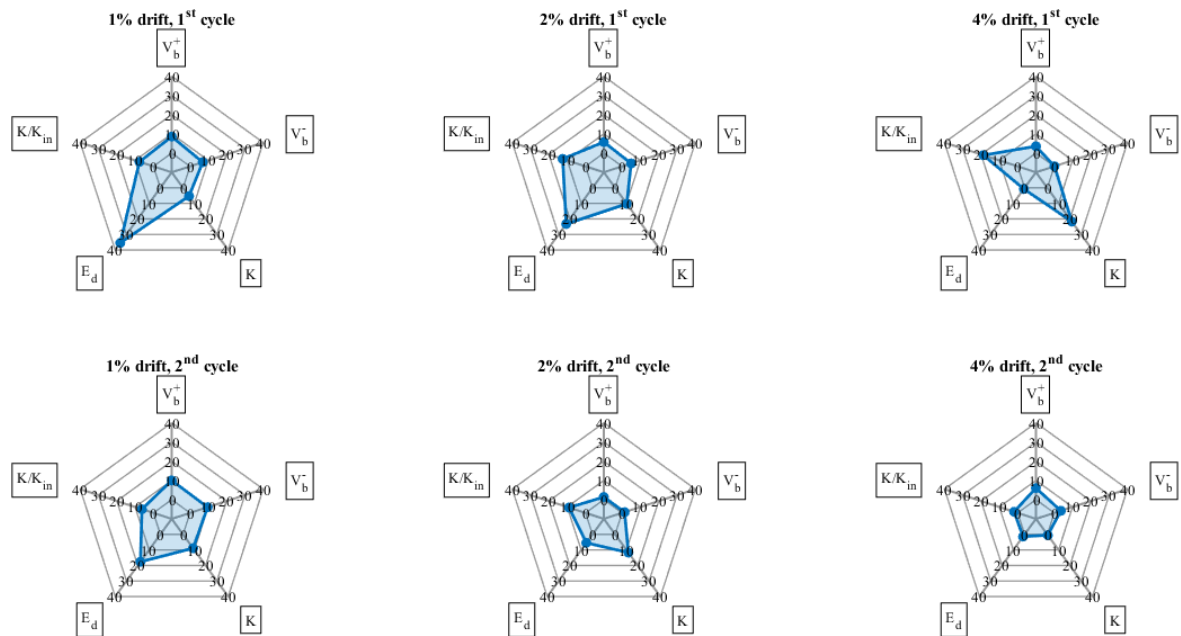


Figure B.4. Absolute errors for control quantities, Park and Paulay specimen.

## CONCLUSIONS

This Appendix presented additional simulations of quasi-static cyclic tests on RC columns applying the modeling approach discussed in Chapter 4. Six benchmark specimens have been retrieved from [1]. The comparison between numerical and experimental response revealed strong agreement both in terms of  $V_b$  and hysteretic response. Four control quantities have been considered to provide a quantitative validation of the modeling strategy, confirming the global agreement with experimental data. Finally, further investigation was carried out by computing the absolute errors for  $V_b$ ,  $K$ ,  $E_d$  and  $K/K_{in}$  for three drift levels considering two different cycles. Errors tend to be higher for lower drift levels and lower for higher levels, but it is worth pointing out that the absolute values of control quantities for the former are relatively small with respect to the global response and consequently differences are almost negligible in absolute terms. The simulations confirm the effectiveness of the modeling strategy for RC members in pristine conditions, providing a realistic simulation of the response.

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## **APPENDIX C: IMPLICATIONS OF DIFFERENT CORROSION DAMAGE MODELING STRATEGIES ON LONG-TERM SEISMIC PERFORMANCE ASSESSMENT OF RC BRIDGES**

*This appendix is extensively based on the manuscript Reale, S., Todaro, M., Furinghetti, M., Pavese, A., & Palermo, A. (2026). "Implications of different corrosion damage modeling strategies on long-term seismic performance assessment of RC bridges". Procedia Structural Integrity, 78, 1657-1664.*

### **INTRODUCTION**

Harsh environmental exposure conditions can lead to severe deterioration phenomena on RC structures, including bridges. Corrosion represents the main deterioration phenomenon in RC members, leading to non-constant performance of structures over time and including a number of facets. Corrosion negatively affects several aspects, such as reinforcement section, steel and concrete mechanical properties, confinement level, cyclic bond capacity, shift of failure mode from ductile to brittle and can lead to concrete cracking and spalling due to the expansion of corrosion products. Given the complexity of the phenomenon, predictive modeling of the long-term performance of RC structures requires carefully considering corrosion-induced deterioration in order to provide realistic and reliable results. Significant research effort has been dedicated to this issue in the past. Alipour et al. [1] studied the long-term performance of deteriorating RC bridges considering corrosion-induced deterioration through a relatively simplified approach including uniform reinforcement section loss and steel yield strength reduction. Shekhar et al. [2] provided recommendations for modeling aging bridges exposed to chlorides, discussing uniform and localized steel area loss, loss of cover and core concrete strength and reduction of steel yielding and ultimate strength. Xu et al. [3] discussed the numerical modeling of deteriorating shear-critical columns. Dizaj and Kashani [4] proposed and validated a modeling technique including critical aspects such as Low-Cycle Fatigue (LCF), inelastic buckling and bond-slip. Reale et al. [5] proposed a refined and practical corrosion damage modeling strategy, validated it against experimental tests carried out on corroded bridge piers and presented an application discussing the long-term seismic performance assessment of an RC bridge [6].

Given that the numerical model development represents one of the key modules of the Performance Based Earthquake Engineering (PBEE) framework, the assumptions related to the level of detail of corrosion damage modeling affect the results of seismic assessment. The present contribution investigates the implications of progressively more refined corrosion damage modeling strategies on the long-term seismic performance assessment of RC bridges. The extensive numerical investigation is carried out following state of the art PBEE principles and considering a prototype bridge exposed to chloride attack. Chloride diffusion and corrosion propagation are characterized probabilistically through Monte Carlo simulation, accounting for the inherent uncertainty of the process. The results of the seismic assessment are systematically scrutinized through fragility curves and failure rates related to four damage states.

### CORROSION DAMAGE MODELING STRATEGIES

Four progressively more detailed corrosion damage modeling strategies have been considered for the present research, as summarized in Table C.1. Strategy 1 only includes reinforcement section loss to model the effects of corrosion. Steel section loss is modeled according to [7]. Assuming hemispherical pits [8], the residual area in presence of pitting  $A_{R,P}$  and the pit width  $a(t)$  can be estimated as per Equations (C.1)-(C.4):

$$(C.1) \quad a(t) = 2p(t) \sqrt{1 - \left(\frac{p(t)}{D_0}\right)^2}$$

$$(C.2) \quad A_{R,P} = \begin{cases} \frac{\pi D_0^2}{4} - A_1 - A_2 & \text{for } p(t) \leq \frac{\sqrt{2}}{2} D_0 \\ A_1 - A_2 & \text{for } \frac{\sqrt{2}}{2} D_0 < p(t) \leq D_0 \\ 0 & \text{for } p(t) > D_0 \end{cases}$$

$$(C.3) \quad A_1 = \frac{1}{2} \left[ 2 \arcsin \left( \frac{a(t)}{D_0} \right) \left( \frac{D_0}{2} \right)^2 - a(t) \left| \frac{D_0}{2} - \frac{p(t)}{D_0} \right| \right]$$

$$(C.4) \quad A_2 = \frac{1}{2} \left[ 2 \arcsin \left( \frac{a(t)}{2p(t)} \right) \cdot p(t)^2 - a(t) \cdot \frac{p(t)^2}{D_0} \right]$$

Where  $D_0$  is the pristine bar diameter and  $p(t)$  is the maximum pit depth. The residual area in a given pit location  $A_{C,P}$  can be estimated according to Equation (C.5)[7]:

$$(C.5) \quad A_{C,P} = \left[ 1 - \frac{a(t)}{2D_0} \right] \left[ A_{C,U}(t) - \frac{\pi D_0^2}{4} \right] + A_{R,P}(t)$$

Strategy 2 introduces the reduction of steel yield strength  $f_y$ , ultimate strength  $f_u$ , cover concrete compressive strength  $f_c$  and core concrete compressive strength  $f'_c$ . Deterioration models developed by Du et al. [9] have been applied to estimate the time-variant values of  $f_y$  and  $f_u$ :



$$(C.6) \quad f_{y,c} = (1 - 0.005\varphi)f_{y,0}$$

$$(C.7) \quad f_{u,c} = (1 - 0.005\varphi)f_{u,0}$$

In Equations (C.6)-(C.7),  $f_{y,0}$  and  $f_{y,c}$  are the yield strength in pristine and corroded condition,  $f_{u,0}$  and  $f_{u,c}$  are the ultimate strength in pristine and corroded condition, and  $\varphi$  is the percentage section loss of corroded reinforcement.  $\varphi$  can be estimated as per Equation (C.8):

$$(C.8) \quad \varphi = \frac{A_0 - A_{s,c}}{A_0} \cdot 100 [\%]$$

Where  $A_0$  and  $A_{s,c}$  are respectively the reinforcement area in pristine and corroded condition.

The reduction of  $f_c$  due to microcracks caused by the expansion of corrosion products is included through the model reported in [10]. The time varying value of  $f_c$  is computed through Equation (C.9):

$$(C.9) \quad f_{c,corr}(t) = \frac{f_c}{1 + 0.1 \cdot \left( \frac{\varepsilon_1(t)}{\varepsilon_{c0}} \right)}$$

where  $\varepsilon_{c0}$  is the strain at peak compressive stress and  $\varepsilon_t$  is the average tensile strain in cracked concrete.  $\varepsilon_t$  can be computed as per Equation (C.10), based on [11]:

$$(C.10) \quad \varepsilon_1(t) = \frac{n_{bars} \sum_i 2\pi(v_{rs}-1)d_{rs,i}(t)}{b_0}$$

where  $n_{bars}$  is the number of compressed bars,  $b_0$  is the section width in pristine condition,  $v_{rs}$  is the ratio of volumetric expansion of oxides and  $d_{rs,i}(t)$  is the depth of corrosion attack on the  $i$ -th bar.

The reduction of  $f'_c$  due to the reduced confinement level  $f'_l$  is modeled according to [12]:

$$(C.11) \quad f'_c = f_{c0} \left( 2.254 \sqrt{1 + \frac{7.94 f'_l}{f_c}} - \frac{2 f'_l}{f_c} - 1.254 \right)$$

Strategy 3 includes reduction of steel ultimate strain  $\varepsilon_{su}$  [13] and core concrete ultimate strain  $\varepsilon_{cu}$  [12]:

$$(C.12) \quad \varepsilon_{su,c} = (1 - 0.05\varphi)\varepsilon_{su,0}$$

$$(C.13) \quad \varepsilon_{cu}^c = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{su}^c}{f'_c}$$

In Equations (C.12)-(C.13),  $\varepsilon_{su,0}$  and  $\varepsilon_{su,c}$  are the steel ultimate strain in pristine and corroded condition,  $\varepsilon_{cu}$  is the core concrete ultimate strain in corroded condition,  $\rho_s$  is the volumetric transverse reinforcement ratio of confined core,  $f_{yb}$  is the yield strength of transverse reinforcement and  $\varepsilon_{st}^c$  is the ultimate strain of corroded transverse reinforcement.

Strategy 4 includes bond-slip and LCF. Bond-slip, included in the model through a zero-length element, is described according to [14]. Corrosion is included by modifying the slip at yield stress  $s_y^c$  according to:

$$(C.14) \quad s_y^c(mm) = 2.54 \left( \left( \frac{d_b^c(mm)}{8437} \right) \frac{f_{y,c}(MPa)}{\sqrt{f_c'(MPa)}} (2\alpha + 1) \right)^{\frac{1}{\alpha_s}} + 0.34$$

where  $d_b^c$  is the residual diameter of a bar at time t and  $\alpha_s$  is a parameter taken as 0.4. The effects of corrosion on LCF are considered by modifying the  $\alpha$  factor of Coffin-Manson fatigue model as per [15]:

$$(C.15) \quad \alpha_c = \alpha[1 + 0.004\varphi]$$

**Table C.1. Overview of the corrosion damage modeling strategies applied for the long-term seismic performance assessment.**

| Modeling Strategy | Reinforcement Section Loss | $f_y$ | $f_u$ | $\varepsilon_{su}$ | $f_c$ | $f_c'$ | $\varepsilon_{cu}$ | Bond-Slip | LCF |
|-------------------|----------------------------|-------|-------|--------------------|-------|--------|--------------------|-----------|-----|
| 1                 | Yes                        | No    | No    | No                 | No    | No     | No                 | No        | No  |
| 2                 | Yes                        | Yes   | Yes   | No                 | Yes   | Yes    | No                 | No        | No  |
| 3                 | Yes                        | Yes   | Yes   | Yes                | Yes   | Yes    | Yes                | No        | No  |
| 4                 | Yes                        | Yes   | Yes   | Yes                | Yes   | Yes    | Yes                | Yes       | Yes |

## CASE STUDY

The present investigation considers a prototype bridge (Bridge 2 in Table 1.1 and Table 1.2) designed according to modern design and durability standards. The structure is located in Reggio Calabria (Italy). The exposure conditions are assumed to be comparable to the XS3 Eurocode exposure class. The bridge consists of two 30 m simply supported spans supported by elastomeric devices. The deck is composed of six PC beams with a 0.25 m thick RC slab. The bridge bent includes three columns with 0.9x0.9 m section and a cap beam. The longitudinal reinforcement of the columns consists of 36  $\varphi 24$  bars, with four legs  $\varphi 10$  stirrups with spacing 150 mm as transverse reinforcement. The concrete cover depth is 60 mm, according to Eurocode prescriptions. A water-to-cement ratio  $w/c$  of 0.45 has been assumed.

## LONG TERM SEISMIC PERFORMANCE ASSESSMENT

### Corrosion Modeling

Time dependent chloride concentration  $C(x, t)$  at a given depth  $x$  and time  $t$  is estimated through the error function solution of 1D Fick's Second Law of Diffusion [16], as per Equation (C.16):

$$(C.16) \quad C(x, t) = C_0 + (C_{s, \Delta x} - C_0) \cdot \left[ 1 - \operatorname{erf} \left( \frac{x - \Delta x}{2 \cdot \sqrt{D_{app}(t) \cdot t}} \right) \right]$$

Where  $C_0$  is the initial chloride content [wt-%/c],  $C_{s, \Delta x}$  is the chloride content resulting from environmental exposure at depth  $\Delta x$ ,  $\Delta x$  is the depth of the convection zone [mm],  $\operatorname{erf}$  is the Gaussian error function and  $D_{app}(t)$  is the apparent chloride diffusion coefficient at time  $t$  [mm<sup>2</sup>/year]. Corrosion is assumed to start once that the critical concentration of free chloride ions  $C_{crit}$  on the surface of steel bars is reached. Due to the inherent uncertainty of the process, the parameters of Equation (C.16) are modeled as Random Variables (RVs), as described in Table C.2, and Monte Carlo simulation is performed to model chloride diffusion and corrosion propagation over time.

Table C.2. RVs used in the simulation, based on [16].

| RV                                                | Distribution | Mean ( $\mu$ ) | Std ( $\sigma$ ) | Bounds                     |
|---------------------------------------------------|--------------|----------------|------------------|----------------------------|
| $D_{rem,0}$ [10 <sup>-12</sup> m <sup>2</sup> /s] | Normal       | 4.5            | 0.2 $\mu$        | $\geq 0$                   |
| $\alpha$ [-]                                      | Normal       | 0.4            | 0.16             | $0 \leq \alpha \leq 1$     |
| $T_{real}$ [K]                                    | Normal       | 292            | 5                | -                          |
| $b$                                               | Normal       | 4800           | 700              | -                          |
| $C_{s, \Delta x}$ [wt-%/c]                        | Lognormal    | 4              | 1.8              | $\geq 0$                   |
| $\Delta x$ [mm]                                   | Beta         | 10             | 5                | $0 \leq \Delta x \leq c-1$ |
| $C_{crit}$ [wt-%-c]                               | Beta         | 0.6            | 0.15             | $0.2 \leq C_{crit} \leq 2$ |
| $c$ [mm]                                          | Normal       | $c_{nom}$      | 6                | $\geq \varphi_i + 1$       |
| $f_c$ [MPa]                                       | Lognormal    | 43             | 5                | $\geq 0$                   |
| $f_y$ [MPa]                                       | Lognormal    | 495            | 30               | $\geq 0$                   |

The corrosion propagation phase can be described following Faraday's Law [17]:

$$(C.17) \quad p(t_p) = 0.0116 \cdot R \cdot t_p \cdot i_{corr}(t_p)$$

In Equation (C.17),  $R$  is the pitting factor,  $t_p$  is the propagation period and  $i_{corr}(t_p)$  is the corrosion current density [ $\mu\text{A}/\text{cm}^2$ ].  $R$  is sampled from a Gumbel distribution [18], while the time-dependent value of  $i_{corr}$  can be estimated as per Equation (C.18) [19]:

$$(C.18) \quad i_{corr}(t_p) = \frac{37.8 \cdot \left(1 - \frac{w}{c}\right)^{-1.64}}{x} \cdot 0.85 \cdot t_p^{-0.29} [\mu\text{A}/\text{cm}^2]$$

### Ground Motion Selection

The distribution of the Engineering Demand Parameters (EDPs) conditioned on the Intensity Measure (IM) level is estimated through Multiple Stripe Analysis (MSA) [20] considering 10 IM levels with increasing return period  $T_R$  of 10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 years. The spectral acceleration corresponding to the bridge fundamental period  $S_a(T_l)$  has been selected as IM. The site hazard curve has been computed using REASSESS [21] assuming a  $V_{s,30}$  of 300 m/s. Seismic hazard deaggregation has been performed at each IM level, and hazard consistent ground motions have been selected through the Conditional Spectrum method [22][23]. For each IM level, 30 pairs of ground motions have been retrieved from the PEER database [24] matching mean and variance of the target  $S_a$  distribution.

### Numerical Modeling

The numerical model of the case study bridge has been developed using OpenSees [25]. Elastic beam elements have been used to model the deck, according to the common assumption of linear elastic response of the superstructure in the presence of seismic events. Bearing devices have been modeled through *ElastomericBearingPlasticity* elements. The bent cap has been described through elastic beam elements. Bent columns have been modeled using inelastic frames with force-based formulation and Gauss-Radau integration scheme. Fiber-based sections are used. Cover concrete is modeled through *Concrete01* uniaxial material, while *Concrete04* is used for the core concrete. *Steel02* is used to describe the reinforcement fibers, combined with a *MinMax* criterion to simulate bar fractures and the *Fatigue* wrapper to include LCF effects. Zero length elements combined with the *ElasticPPGap* material have been used to model the interaction between deck and abutment backwall, following the indications reported in [26]. Geometrical nonlinearities have been included in the model.

### Fragility Analysis

Four Damage States (DS), namely Slight (DS1), Moderate (DS2), Extensive (DS3) and Complete (DS4) Damage, have been considered. The thresholds used in the fragility analysis have been reported in Table C.3, based on [27]. Fragility curves for each DS have been obtained through the Maximum Likelihood Estimation (MLE) method [28]. The total dispersion of each fragility curve  $\beta_{tot}$  is estimated as per Equation (C.19):

$$(C.19) \quad \beta_{tot} = \sqrt{\beta_{gm}^2 + \beta_{mod}^2 + \beta_{ds}^2}$$

Where  $\beta_{gm}$  is the dispersion component associated with record-to-record variability,  $\beta_{mod}$  is the component associated with modeling uncertainty and  $\beta_{ds}$  is the uncertainty in the DS definition.  $\beta_{gm}$  is estimated through the MLE method,  $\beta_{mod}$  is computed according to the FEMA P-58 [29] and  $\beta_{ds}$  is defined as per [30]. System fragility functions  $P(F_{system})$  have been defined based on the probability of exceeding a given DS for the  $i$ -th component  $P(F_i)$  taking the upper bound of Equation (C.20), as per [31]:

$$(C.20) \quad \max_{i=1}^{N_{comp}} [P(F_i)] \leq P(F_{system}) \leq 1 - \prod_{i=1}^{N_{comp}} [1 - P(F_i)]$$

The effects of different corrosion damage modeling strategies can be efficiently discussed by considering the reduction of the IM value corresponding to the median of the system fragility curve for DS4, as reported in Table C.4. It can be observed that the values tend to decrease over time, in significantly more pronounced way as more details are included in the corrosion damage modeling strategies. The divergence between the four strategies increases as time passes. While for  $t=20$  years the discrepancy is not pronounced, from  $t=40$  years onwards the difference between the strategies becomes not negligible, with relevant effect on the subsequent phases of seismic risk assessment.

**Table C.3. Thresholds for each component and for each DS.**

| Component | EDP               | DS1        | DS2                                      | DS3                                      | DS4           |
|-----------|-------------------|------------|------------------------------------------|------------------------------------------|---------------|
| Piers     | Displacement      | $\Delta_y$ | $\Delta_y + 0.5(\Delta_{ir} - \Delta_y)$ | $\Delta_y + 2/3(\Delta_{ir} - \Delta_y)$ | $\Delta_{ir}$ |
| Bearings  | Shear deformation | 150%       | 200%                                     | 300%                                     | Unseating     |
| Backwall  | Displacement      | $d_{gap}$  | $d_{y,ab}$                               | $\Delta_y + 2/3(\Delta_{ir} - \Delta_y)$ | $d_{n,ab}$    |

**Table C.4. Evolution of DS4 median IM values for each modelling strategy. Values between branches represent the ratio between the median IM value at a given time and its initial value.**

| Modelling Strategy | Initial   | 20 years      | 40 years       | 60 years      | 80 years      |
|--------------------|-----------|---------------|----------------|---------------|---------------|
| 1                  | 0.638 (-) | 0.638 (1.000) | 0.634 (0.993)  | 0.634 (0.993) | 0.631 (0.989) |
| 2                  | 0.638 (-) | 0.634 (0.993) | 0.6151 (0.964) | 0.594 (0.930) | 0.583 (0.913) |
| 3                  | 0.638 (-) | 0.617 (0.967) | 0.583 (0.914)  | 0.573 (0.898) | 0.557 (0.873) |
| 4                  | 0.615 (-) | 0.594 (0.967) | 0.558 (0.908)  | 0.521 (0.847) | 0.504 (0.820) |

### Seismic Risk Assessment

Seismic Risk Assessment is performed by computing the failure rate  $\lambda_f$  according to:

$$(C.21) \quad \lambda_f = \int_0^\infty P(F_{system}|IM = x)|d\lambda(IM > x)|$$

Where  $|d\lambda(IM > x)|$  is the absolute value of the derivative of the site hazard curve. For consistency, the discussion will refer to DS4. Figure C.1 reports the evolution over time of  $\lambda_f$  and the normalized values with respect to the initial  $\lambda_f$  value.  $\lambda_f$  values computed applying Strategy 1 are almost constant over time, due to the oversimplified representation of corrosion damage. The seismic risk associated with Strategy 4 is higher also for the initial time due to the inclusion of bond-slip effects. Risk values through Strategies 2, 3 and 4 denote an increase of the seismic risk metric up to around 1.2, 1.36 and 1.66 times the initial value. It is therefore clear that Strategy 1 is not suitable for long-term seismic performance assessment of RC bridges and that a more detailed description of corrosion damage is associated with more pronounced effects on the long-term seismic assessment. It is worth noting that the results have been obtained by the analysis of a modern bridge and that the same exercise repeated on existing bridges may lead to even more pronounced discrepancies between the different approaches. The effects of corrosion on phenomena such as LCF and bond-slip still represent open research questions, possibly hindering the application of Strategy 4. The assumptions on the level of detail of corrosion damage modeling are fundamental for the assessment, highlighting the need for additional research.

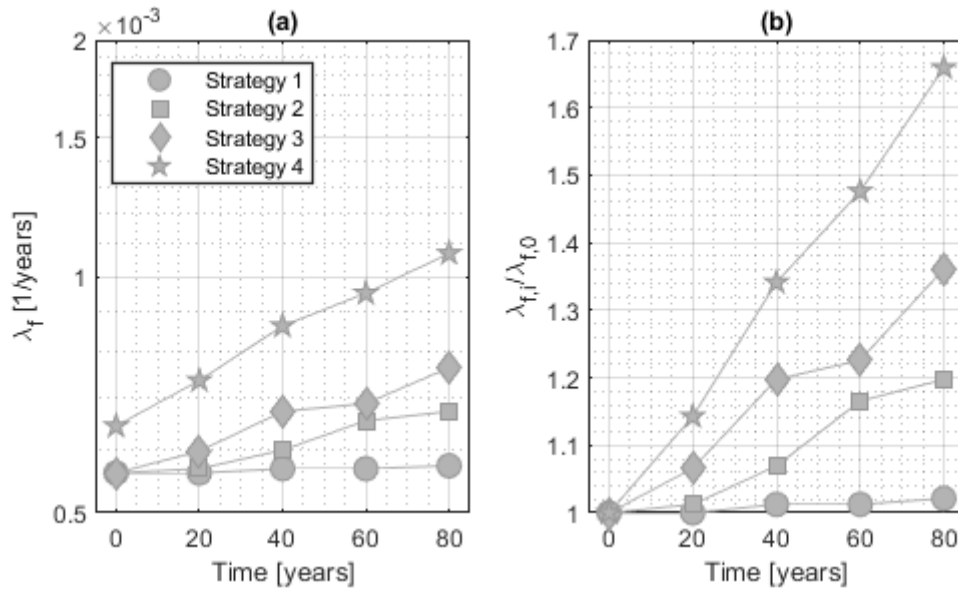


Figure C.1. (a)  $\lambda_f$  values and (b) normalized  $\lambda_f$  values over time for the four corrosion damage modeling strategies.

## CONCLUSIONS

This contribution investigated the implications of four progressively more refined corrosion damage modeling strategies on the long-term seismic performance assessment of RC bridges considering a case study bridge exposed to chloride attack. The assessment has been carried out according to the PBEE principles. Corrosion evolution over time has been characterized through Monte Carlo simulation accounting for the inherent uncertainty of the process. The results revealed that the median IM values of the fragility curves tend to decrease over time, in a more pronounced way as more details are included in the corrosion damage modeling strategies. Risk values computed through Strategy 1 are almost constant over time, while Strategy 2, 3 and 4 denote an increase of the seismic risk metric up to around 1.2, 1.36 and 1.66 times the initial value. Strategy 1 appears to not be suitable for long-term seismic performance assessment of RC bridges. The results confirm that a more detailed description of corrosion damage is associated with more pronounced effects on the long-term seismic assessment, but additional research is needed on this issue given that the practical application of more refined strategies may be hindered by the lack of experimental validation.

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## APPENDIX D: COMPLETE SET OF DDBA VALIDATION RESULTS

### D/C RATIOS FOR BRIDGE 2 IN SCENARIOS 1 AND 2

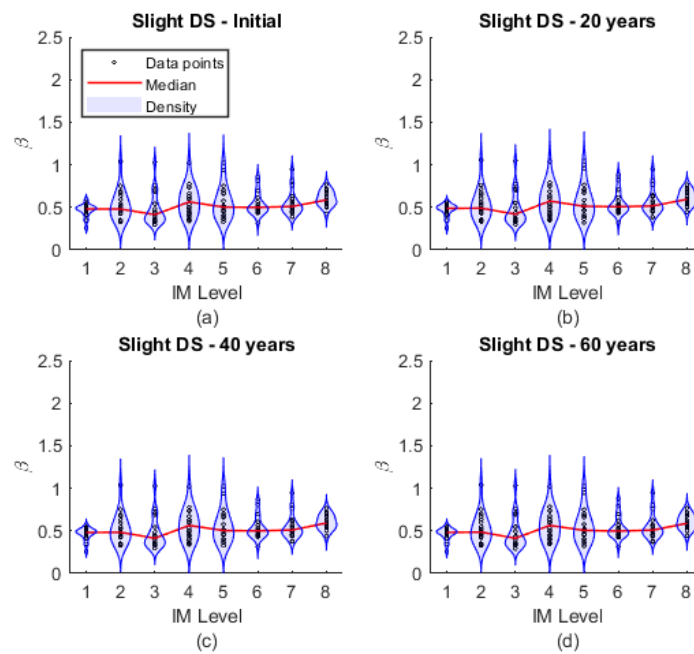


Figure D.1.  $\beta$  ratios for Slight Damage State, for  $t=0, 20, 40, 60$  years. XS1 Class. Bridge 2.

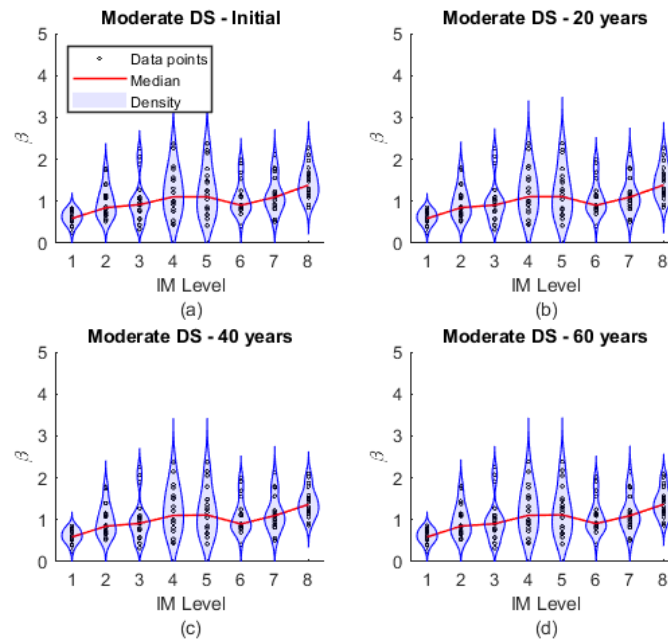


Figure D.2.  $\beta$  ratios for Moderate Damage State, for  $t=0, 20, 40, 60$  years. XS1 Class. Bridge 2.

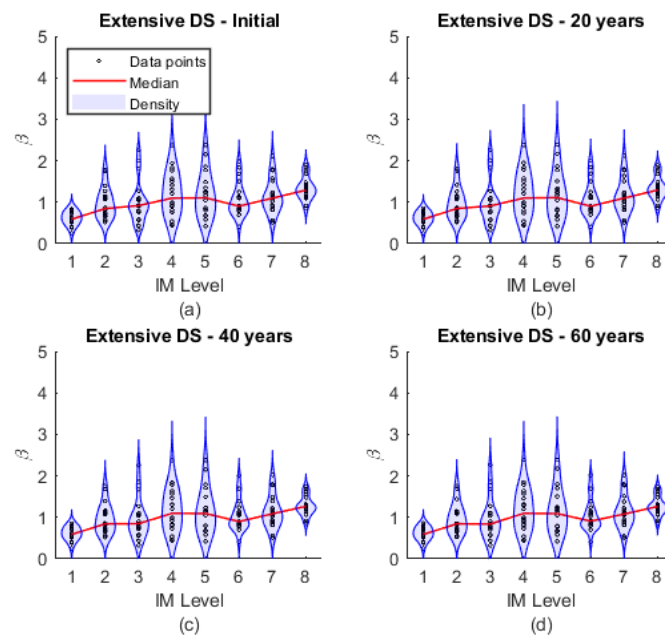


Figure D.3.  $\beta$  ratios for Extensive Damage State, for  $t=0, 20, 40, 60$  years. XS1 Class. Bridge 2.

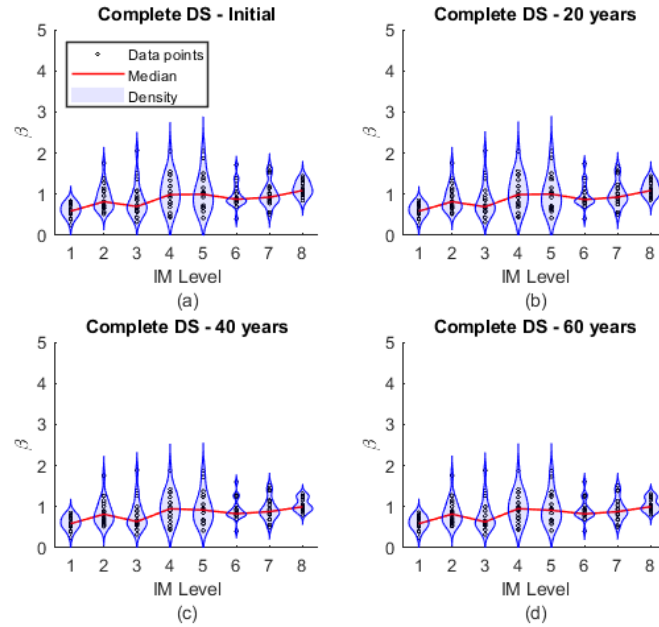


Figure D.4.  $\beta$  ratios for Complete Damage State, for  $t=0, 20, 40, 60$  years. XS1 Class. Bridge 2.

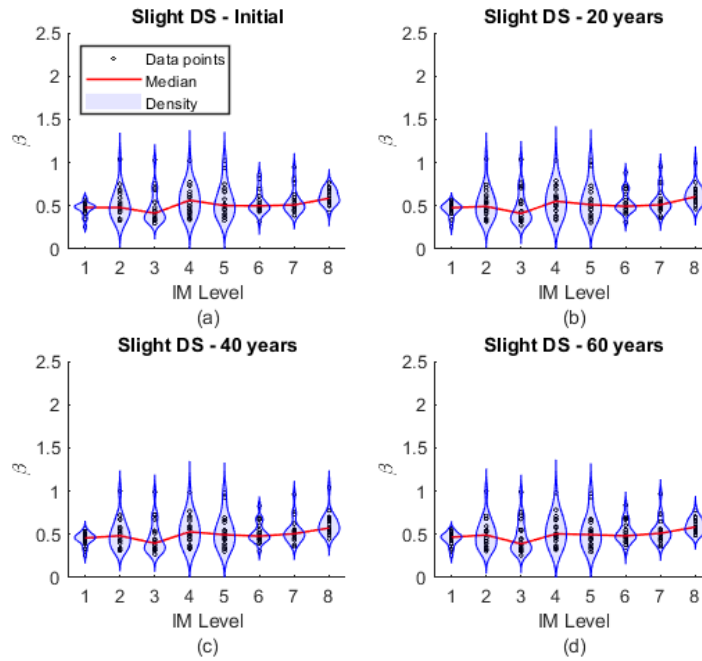


Figure D.5.  $\beta$  ratios for Slight Damage State, for  $t=0, 20, 40, 60$  years. XS2 Class. Bridge 2.

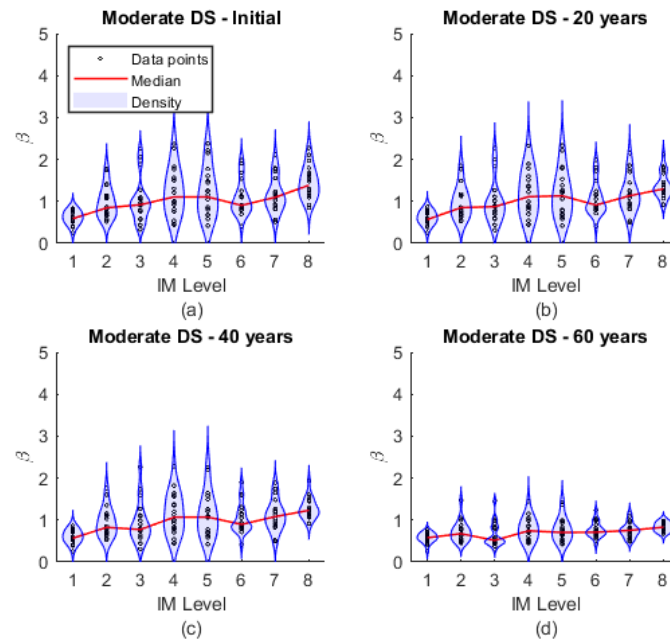


Figure D.6.  $\beta$  ratios for Moderate Damage State, for  $t=0, 20, 40, 60$  years. XS2 Class. Bridge 2.

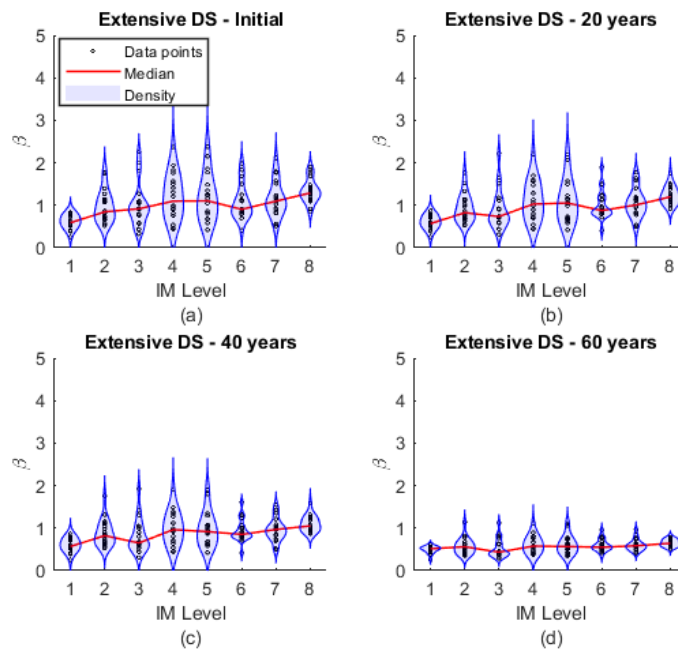


Figure D.7.  $\beta$  ratios for Extensive Damage State, for  $t=0, 20, 40, 60$  years. XS2 Class. Bridge 2.

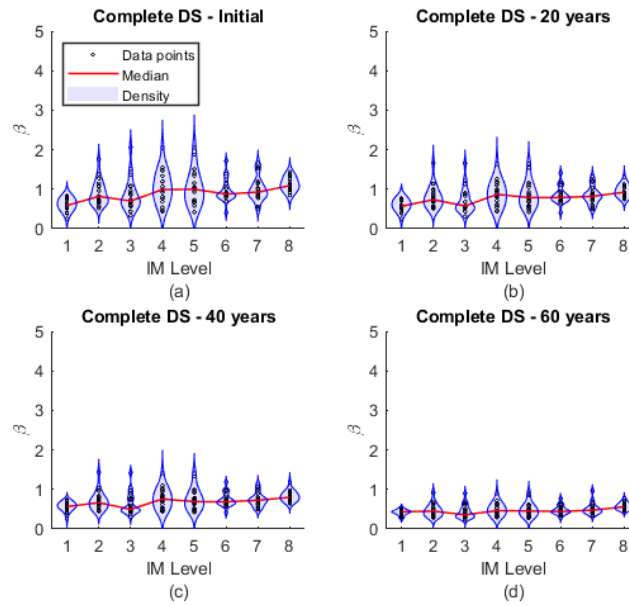


Figure D.8.  $\beta$  ratios for Complete Damage State, for  $t=0, 20, 40, 60$  years. XS2 Class. Bridge 2.