Maintenance interventions period of existing RC motorway viaducts located in moderate/high seismicity zones

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Abstract

Considering the recent collapses which have affected several existing reinforced concrete (RC) motorway bridges, the correct maintenance interventions planning of these strategic infrastructures has acquired increasing importance to maintain a suitable safety level even in case of seismic loads. One of the most important problems which influences the structural performance of existing RC motorway bridges is the presence of corrosion which may affecte the bearing-capacity members. In particular, the corrosion of steel reinforcement of the piers due to the carbonation phenomenon is strictly related to the seismic performance of these structures as the viaducts of the Italian motorway networks realized around the 1970's.
In this paper, the evaluation of the seismic vulnerability of three existing RC motorway bridges has been developed using non-linear time history analysis. Three corrosion scenarios (slight, moderate and high) have been analysed. The effects of the carbonation, expressed in terms of piers steel rebars area reduction, have been considered as a function of the age of the viaduct. Risk indices calculated as the ratio of the peak ground acceleration leading to collapse of the first structural element and the design peak ground acceleration, are useful to optimize the scheduling of the motorway networks maintenance interventions.

Keywords*: reinforced concrete structures, bridges, corrosion phenomena, seismic vulnerability, non-linear time history analysis, existing structures*.

Please cite this document as: M. Zucca, P. Crespi, F. Stochino, M. L. Puppio, M. Coni, Maintenance interventions period of existing RC motorway viaducts located in moderate/high seismicity zones, *Structures*, 2023, 47: 976-990. https://doi.org/10.1016/j.istruc.2022.11.135

1. Introduction

During last decades, the occurrence of several collapses which involved existing reinforced concrete (RC) motorway bridges led to considerable attention on the correct planning of the maintenance interventions to achieve a suitable safety level [1-8]. An important aspect that affects the seismic performance of existing RC bridges is the presence of corrosion in the structural elements due to carbonation phenomenon which can lead to a reduction of the piers steel reinforcements area [9-12]. Several Italian RC viaducts were built around 1970's and, consequently, require a series of maintenance interventions to guarantee an adequate safety level [13-15]. It is important to highlight that these structures were mainly designed to resist to vertical loads and only limited horizontal actions. Many important construction details were not implemented in existing RC bridges as the stirrups spacing that characterize the piers that is quite large in comparison to the requirements for modern seismic designed structures.

To evaluate the seismic vulnerability of RC structures different approaches have been proposed especially based on non-linear analysis methods. One of the most important method is the pushover analysis which, however, requires that the dynamic behaviour of the analysed structure is characterized by a predominant translational vibration mode [16]. For this reason [17,18] suggested the use of the multi-modal pushover approach. This technique was initially used to analyse the seismic behaviour of unsymmetrical-plan buildings [19,20] but it is a promising approach also to estimate the seismic response of existing RC motorway viaducts. Different probabilistic methods capable of obtaining the fragility curves that characterize the seismic vulnerability of these structures have been proposed in the literature [21-25].

Another recent approach for the evaluation of the seismic performance of existing RC bridges is represented by Incremental Dynamic Analysis (IDA) developed by [26]. A simple alternative of the IDA is the IMPAB method proposed by [27], based on the use of Incremental Modal Pushover Analysis which allows a better representation of seismic input with respect to the standard selection of accelerograms to be scaled at different intensities, usually adopted in IDA [28].

A good compromise between the accuracy of results and the computational efforts is represented by the Non-Linear Time History Analysis (NTHA). In this case the correct choice of the seismic signals, the modelling of strength/stiffness degradation and the evaluation of the evolution of the damping play a role of primary importance. NTHA is one of the most used analysis methods thanks also to the technological evolution of computers and the software for numerical analyses.

Carbonatation and steel corrosion are recurring cause of degradation of exposed RC structures. Recent research aims to investigate the effects of environmental aggressive environment [29] and aging effects finalized to life cycle assessment of bridges [30]. The topic is of significance relevance for the spread of this kind of infrastructure and the difficult to reveal this kind of degradation without specifical tests.

A simplified method with reasonable computational cost that can evaluate the effect of corrosion on existing RC viaducts is currently missing in the literature.

In this paper, the influence of the carbonation induced corrosion on the seismic behaviour of existing RC motorway viaducts is taken into account by considering piers steel reinforcements diameter reduction. Special attention is devoted to the evaluation of the maintenance schedule necessary to have a suitable safety level. The maintenance time intervention is calculated starting from risk indices of the analysed structures they are expressed as peak ground acceleration (PGA) and corresponding return period (RP), coming from the execution of a series of NTHA taking into account (i) the ductile and (ii) the brittle failure mechanism of the viaduct piers. Three different corrosion scenarios (slight, moderate and high) are taken into account during the service life of three existing RC bridges located in Northern Italy until 75 years from the construction time. The obtained results are analysed considering the correlation between the corrosion effects and the reduction of the time intervention.

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After this brief introduction (Section 1) and the description of the research objective (Section 2) the paper is organised as follows: Section 3 illustrates the corrosion and structural models, while the case study analysis is reported in Section 4. Finally, some conclusive remarks are drawn in Section 5.

2. Research objective

Considering the issues described in previous Section 1, the aim of this research work is to provide an efficient procedure which allows the assessment of the safety level of existing RC bridges subjected to seismic action, based on the implementation of simplified Finite Element models where the structural elements are modelled using only beam elements in order to reduce the computational efforts and where the non-linear behaviour of the piers is considered through the application of appropriate concentrated plastic hinges located at the base of the piers where the formation of the ductile mechanism is hypothesized. This approach was developed to analyse different motorway viaducts such as way as to define the intervention priorities even within the same motorway network in order to have a correct scheduling of maintenance interventions according to the edge of the bridges and considering the effects of the corrosion due to the carbonation on the load-bearing capacity of the analysed structures. In particular, different corrosion scenarios are assumed to evaluate the reduction of the maintenance period as a function of the more or less aggressive environment where the bridge is located.

Furthermore, the results obtained can be used to determine the different structural elements of the bridge that need adequate maintenance interventions in order to guarantee a predetermined safety level of the structure.

3. Corrosion effects and structural modelling

To evaluate the seismic vulnerability of existing RC bridges, simplified Finite Element (FE) models have been implemented using MIDAS Civil software [31] following the approach proposed in [32,33]. In particular the piers, the pier caps and the deck have been modelled using Timoshenko beam elements while the elastomeric bearings have been represented by elastic links with translational and rotational stiffnesses calculated according to [34]. The connection between beam elements representing the pier caps, the deck and the elastic links used to model the elastomeric bearings is obtained through a series of rigid links (Fig.1).

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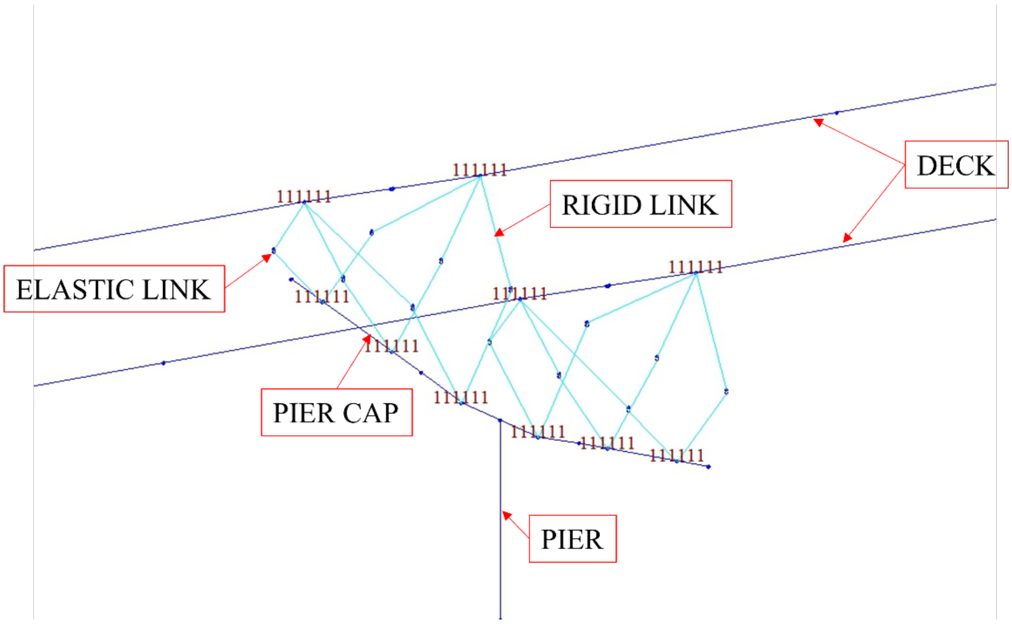
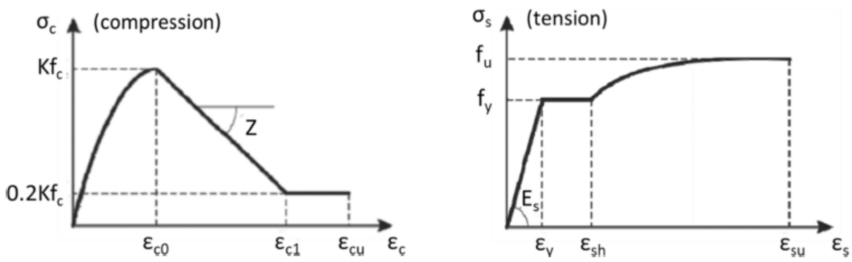


Figure 1. Pier cap-deck connection.

The abutments are modelled as perfect restraints applied to the base node of the bearings on the top of the abutment, while the piers foundations are considered as fully constrained nodes at the base of each pier.
To obtain a more accurate dynamic behaviour, the reduction of the bending stiffness of the gross-section of the piers due to the concrete cracking has been introduced with appropriate scale factors evaluated starting from the moment-curvature diagram (M-χ) that characterizes the gross-section, as reported in [32,35].
Furthermore, the deck stiffness is not reduced because, according to [36], despite the formation of bridge decks cracks, remain in the elastic phase during seismic events. To evaluate the dynamic behaviour of the existing RC viaducts, the contributions of the structural and non-structural masses are taken into account in the Finite Element (FE) model while the presence of the traffic loads is neglected, according to [37].
As mentioned before, two collapse mechanisms of the piers are considered: (i) the ductile mechanism which depends on the moment-curvature diagram of the gross-section and that presents an initial elastic branch by a large strain hardening plastic behaviour and (ii) the brittle mechanism related to the shear resistance of the pier. The ductile collapse mechanism is based on the plastic hinge rotational capacity while the fragile collapse mechanism is ruled by the ultimate shear strength of the considered pier.
The non-linear behaviour of the materials is considered through the following constitutive laws: (i) Kent and Park model [38] for the concrete (Fig.2a) and (ii) Park Strain Hardening model [39] for the steel reinforcement (Fig.2b).



(a) (b)
Figure 2. (a) Kent and Park and (b) Park Strain Hardening constitutive law [38,39].

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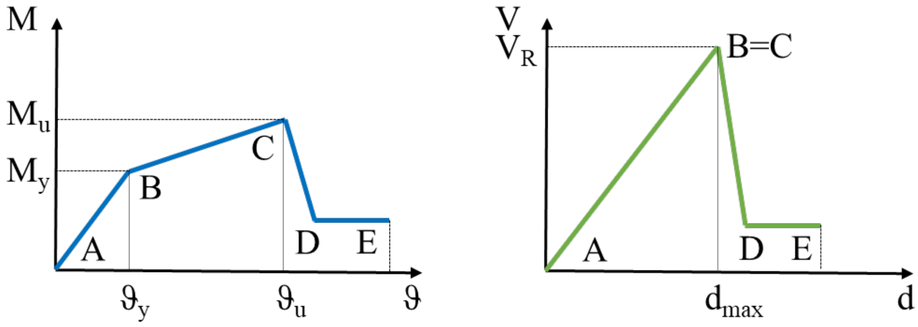
Concentrated plastic hinges, whose characteristics have been calculated as reported in [40,41], have been introduced in the FE model at the base of the piers assuming a cantilever structural behaviour. The considered ultimate limit states are: (i) for the ductile collapse mechanism, the achievement of ¾ of the ultimate rotation ϑ_u (Fig.3a) while (ii) for the brittle collapse mechanism the equality between shear demand and capacity V_R in the considered structural element (Fig.3b). To calculate the value of V_R the formulation proposed in [42] for the cyclic shear resistance has been used, based on the sum of three different terms depending on the axial load, the concrete strength, and the characteristics of the stirrups.



(a) (b)

Figure 3. (a) Ductile mechanism and (b) brittle mechanism.

To introduce in the FE model the corrosion effects due to carbonation, a simplified analytical approach has been used yielding to a progressive reduction of the steel reinforcement diameters [43]. Considering the following Eq.(1), the penetration law in a generic concrete volume is defined by a parabolic trend:

$$s = k \cdot t^{1/n} \tag{1}$$

in which s defines the thickness of the carbonated layer of concrete, t the time, k the penetration rate coefficient and n the parameter which depends on the characteristics of the concrete. As reported in [44], it is possible to take n = 2 considering that the existing RC bridges analysed in this work were built between the 1960s and 1970s using normal compacted concrete. Considering [45], the residual service life (t_{res}) of the bridge is calculated starting from the initiation time (t_i), the propagation time (t_p) and the maximum expected rebar radius reduction (P_{lim}) as shown in Eq.(2).

$$t_{res} = t_i + t_p - t = \left(\frac{c}{k}\right)^2 + \frac{P_{lim}}{i_{corr}} - t \tag{2}$$

where t represents the bridge age, k is the penetration rate coefficient (in mm/years$^{0.5}$), i_{corr} is the mean corrosion current density (in μA/mm^2) and c is the concrete cover thickness. The existing RC bridges considered in this work were designed without considering the presence of the seismic action and for this reason the evaluation of the residual service life (t_{res}) as proposed in Eq.2 is not useful. In fact, the approach used in this work estimates the correlation between the reduction of the steel reinforcement area and the seismic capacity of the bridges, through the following Eqs.(3)-(4):

$$d(t) = d_0 - 2P(t) = d_0 - 2i_{corr}k(t - t_i) \tag{3}$$

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$$A_s(t) = \pi[d_0 - 2i_{corr}k(t - t_i)]^2/4 \quad (4)$$

where d denotes the reduction of the steel reinforcement diameter, d_0 is the initial bar diameter, $P(t)$ represents the corroded thickness.
Furthermore, Eq.(4) reports the relation that regulate the cross-section area variation (A_s).
As mentioned before, three corrosion scenarios have been considered in this work: (i) slight corrosion scenario characterized by a value of i_{corr} = 0.1 μA/cm^2, (ii) moderate corrosion scenario having i_{corr} =1 μA/cm^2 and (iii) high corrosion scenario with i_{corr} = 5 μA/cm^2 [46].
The reduction of the rebar area has been evaluated considering the reduction of the steel rebars diameter as the difference between the initial diameter (d_0) and the diameter referred to the time of interest ($d(t)$). It is important to highlight that this reduction strictly depends on following parameters: (i) the initiation time (t_i) determined as a statistical variable and (ii) the corrosion current density (i_{corr}) obtained considering as reported in the design codes or from the experimental results. Considering an initial value of cover thickness equal to 25 mm, the iterative process has been developed for different concrete type. In this work the cover thickness of the longitudinal reinforcements is calculated considering the presence of the stirrups [47].
The following value of the other parameters has been considered: (i) w/c = 0.6 and (ii) t_i = 13.5 years [45] (Table 1) while the compressive strength of the concrete f_{ck} = 28 MPa, the penetration rate coefficient k = 0.0116 and the steel rebar ultimate deformation $\varepsilon_{u,0}$ = 9 % are taken as constant for the different corrosion levels analysed in this work.

Table 1. Relation between *initiation time* (t_i) and *water/cement* (w/c) ratio [44].

w/c	0.4	0.5	0.6
Probability 99% [years]	61.7	26.3	13.7
Probability 50% [years]	60.4	26.0	13.5

To consider the correlation between the evolution of the corrosion effects due to the carbonation phenomenon and the seismic performance of existing RC bridges, a series of non-linear time history analyses have been carried out. In this approach both ductile and brittle collapse mechanism are present in the same FE model, to consider the interaction between the two different failure mechanisms.

4. Case studies

As mentioned in the previous Sections, three existing RC bridges of Northern Italy, built around 1970's, were considered as case studies. Fig.4 shows the FE models of the three analysed viaducts. Table 2 reports the seismic characteristics of the sites where the three viaducts are located.

Table 2. Seismic characteristics of the sites of the case studies, PGA denotes peak ground acceleration.

	Latitude	Longitude	Soil type according to Eurocode 8	PGA
	[-]	[-]	[-]	[g]
Viaduct 1	43.85432	10.37337	C	0.156
Viaduct 2	43.86324	7.96857	C	0.166
Viaduct 3	43.85739	10.37010	C	0.156

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In particular, the viaduct 1 is composed by two independent adjacent carriageways, characterized by four simply supported 40.00 m spans, and it presents a linear planimetric and altimetric layout. The roadway overall width is equal to 9.86 m and each span of the viaduct is realized by precast concrete slab of three prestressed I girders while the deck concrete slab is characterized by a thickness equal to 20 cm. Each span is supported by 2 x 3 elastomeric bearings positioned into the hammer cap. Three hexagonal hollow RC piers support the deck, having height ranging between 5.06 m and 15.51 m.

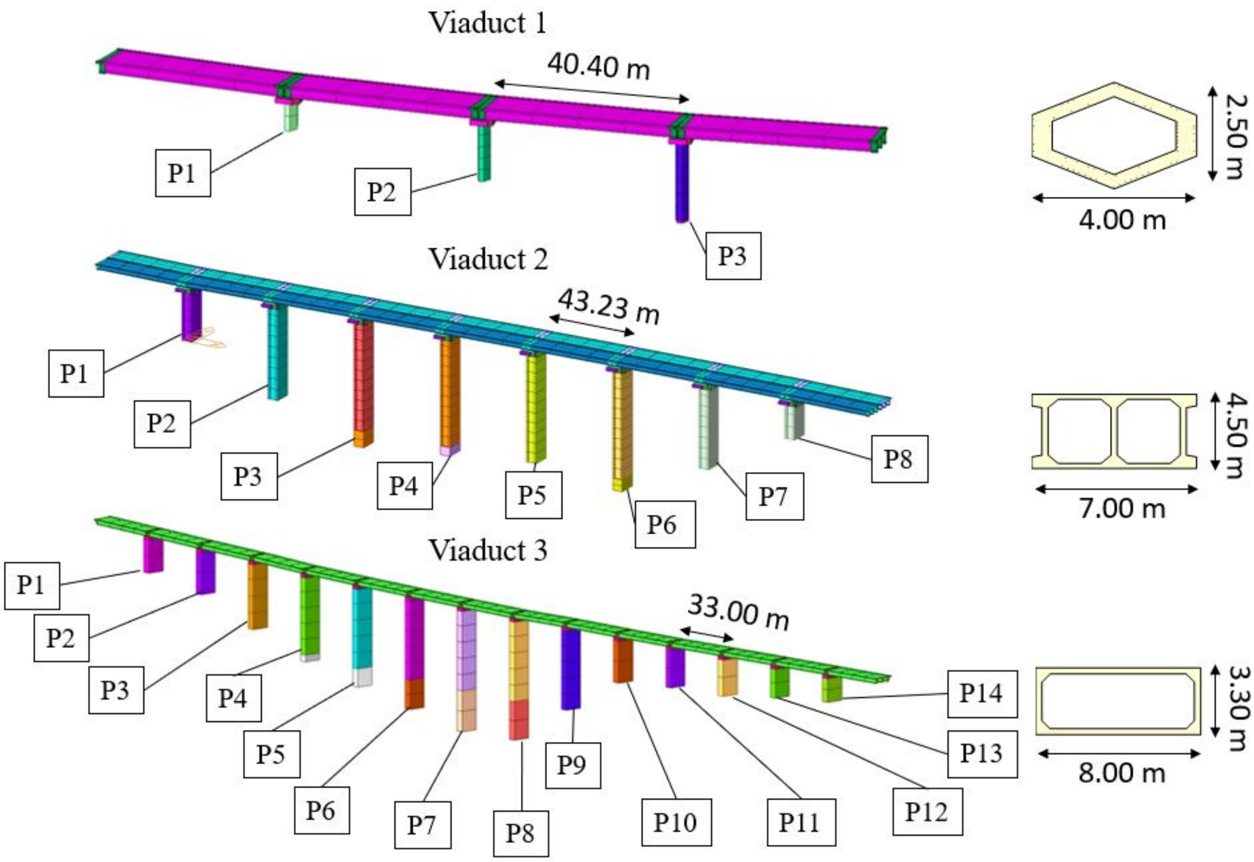

Figure 4. Case studies.

Viaduct 2 is characterized by the presence of single piers supporting the two adjacent and independent carriageways composed by nine simply supported 41.00 m spans. The piers are characterized by a hollow rectangular cross-section and by a height ranging between 14.26 m and 53.02 m. The roadway overall width is about 10.00 m and each span is realized with precast concrete slab supported by three prestressed I girders. Also in this case, the deck is composed by a concrete slab having a thickness equal to 20 cm.

The third viaduct is composed by two adjacent and independent carriageways having fifteen simply supported 34.50 m simply supported spans. The overall width of the roadway is equal to 9.84 m. Each span is realized in precast concrete of three prestressed I girders while the deck concrete slab is characterized by a thickness equal to 25 cm. The piers present a hollow rectangular cross-section with height ranging between 10.46 m and 53.00 m. All the viaducts analysed, have been made with f_{ck} = 28MPa concrete and f_{yk} = 440 MPa steel. The main geometrical and mechanical characteristics of the three case studies are listed in Tables 3 and 4.

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Table 3. Structural characteristics of the three viaducts.

	Spans [n°]	Bridge total length [m]	Elastomeric Bearings [-]	Piers [n°]	Piers shape [-]	Piers Thickness [m]
Viaduct 1	4	200	2 x 3	3	Hexagonal hollow	0.35
Viaduct 2	9	367	2 x 6	8	Rectangular hollow	0.30
Viaduct 3	15	515	2 x 3	14	Rectangular hollow	0.35

Table 4. Piers main characteristics.

Pier	Cross-section dimensions	Height	Longitudinal steel reinforcement	Transverse steel reinforcement
[n°]	[m]	[m]	[-]	[-]
Viaduct 1				
1	4.0 × 2.5	5.06	148Ø14	Ø10/20
2	4.0 × 2.5	10.84	148Ø14	Ø10/20
3	4.0 × 2.5	15.51	148Ø14	Ø10/20
Viaduct 2				
1	7.0 × 4.5	20.63	328Ø16	Ø8/30
2	7.0 × 4.5	39.33	328Ø16	Ø8/30
3	7.0 × 4.5	53.02	328Ø16	Ø8/30
4	7.0 × 4.5	49.82	328Ø16	Ø8/30
5	7.0 × 4.5	46.02	328Ø16	Ø8/30
6	7.0 × 4.5	51.13	328Ø16	Ø8/30
7	7.0 × 4.5	34.40	328Ø16	Ø8/30
8	7.0 × 4.5	14.26	328Ø16	Ø8/30
Viaduct 3				
1	8.0 × 3.3	14.50	164Ø14	Ø10/20
2	8.0 × 3.3	18.50	164Ø14	Ø10/20
3	8.0 × 3.3	28.53	164Ø14	Ø10/20
4	8.0 × 3.3	37.00	164Ø14	Ø10/20
5	8.0 × 3.3	43.00	164Ø14	Ø10/20
6	8.0 × 3.3	48.00	164Ø14	Ø10/20
7	8.0 × 3.3	53.00	164Ø14	Ø10/20
8	8.0 × 3.3	51.97	164Ø14	Ø10/20
9	8.0 × 3.3	34,76	164Ø14	Ø10/20
10	8.0 × 3.3	18.46	164Ø14	Ø10/20
11	8.0 × 3.3	16.31	164Ø14	Ø10/20
12	8.0 × 3.3	15.98	164Ø14	Ø10/20
13	8.0 × 3.3	12.75	164Ø14	Ø10/20
14	8.0 × 3.3	10.46	164Ø14	Ø10/20

Starting from the seismic parameters that characterize the sites where the three existing RC bridges are located, reported in Table 2, seven spectrum-compatible accelerograms are selected for each case study, using Rexel approach [48]. Table 5 summarizes the main characteristics of the seismic inputs obtained from the European Strong-Motion Database (ESD) used in this work while Table 6 shows the first three fundamental natural periods and the related participating masses of the three viaducts where Mass TRAN-LONG. indicates the translational participating mass in the direction of the longitudinal axis of the viaduct while Mass TRAN-TRASV. defines the translational participating mass in transverse direction

Table 5. Seismic records used in this work.

Event	Data Base	Station ID	Year	PGA	PGV	Magnitude M_w
[-]	[-]	[-]	[-]	[m/s^2]	[m/s]	[-]
Viaduct 1 and Viaduct 3						
Umbria Marche	ESD	ST223	1997	0.567	0.048	5.3

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Lazio Abruzzo	ESD	ST152	1984	1.444	0.112	5.9
Ionian	ESD	ST8	1973	2.498	0.255	5.8
Umbria Marche	ESD	ST232	1997	0.501	0.012	5.3
Basso Tirreno	ESD	ST47	1978	1.493	0.083	6.0
Umbria Marche	ESD	ST223	1978	0.326	0.031	5.3
Izmit	ESD	ST3273	1999	1.387	0.089	5.8
Viaduct 2						
Umbria Marche	ESD	ST223	1997	0.567	0.048	5.3
Lazio Abruzzo	ESD	ST152	1984	1.444	0.112	5.9
Ionian	ESD	ST8	1973	2.498	0.255	5.8
Basso Tirreno	ESD	ST47	1978	1.493	0.083	6.0
Umbria Marche	ESD	ST232	1997	0.501	0.012	5.3
Gulf of Corinth	ESD	ST10	1993	0.132	0.011	5.3
Izmit	ESD	ST3272	1999	0.646	0.061	5.8

Table 6. First three natural periods and related participating masses of the case studies.

		Mass TRAN-LONG.	Mass TRAN-TRASV.
Viaduct 1	$t_1 = 1.23$ [s]	0.04%	47.00%
	$t_2 = 1.16$ [s]	66.86%	0.06%
	$t_3 = 0.94$ [s]	0.02%	13.52%
Viaduct 2	$t_1 = 2.75$ [s]	60.42%	0.00%
	$t_2 = 2.63$ [s]	0.00%	37.04%
	$t_3 = 2.48$ [s]	0.00%	9.50%
Viaduct 3	$t_1 = 2.64$ [s]	43.25%	0.00%
	$t_2 = 1.87$ [s]	0.00%	22.44%
	$t_3 = 1.75$ [s]	0.00%	11.86%

Table 7 summarizes the reduction of the longitudinal and transverse steel rebars diameter and area due to the corrosion effects, considering different time intervals since the construction of the viaducts for the different corrosion scenarios defined in Section 3, obtained from the Eq. (3)-(4). The reduction of the steel rebar ultimate deformation (ε_u) is obtained considering as reported in [49]

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Table 7. Diameter and area reduction of the steel rebars.

t	Slight Corrosion Scenario $i_{corr} = 0.1$ [μA/cm²]				Moderate Corrosion Scenario $i_{corr} = 1$ [μA/cm²]				High Corrosion Scenario $i_{corr} = 5$ [μA/cm²]			
	d_0	d	ΔA_s	ε_u	d_0	d	ΔA_s	ε_u	d_0	d	ΔA_s	ε_u
[year]	[mm]	[mm]	[%]	[%]	[mm]	[mm]	[%]	[%]	[mm]	[mm]	[%]	[%]
0-13.5	8.00	8.00	0.00	9.00	8.00	8.00	0.00	9.00	8.00	8.00	0.00	9.00
	10.00	10.00	0.00	9.00	10.00	10.00	0.00	9.00	10.00	10.00	0.00	9.00
	14.00	14.00	0.00	9.00	14.00	14.00	0.00	9.00	14.00	14.00	0.00	9.00
	16.00	16.00	0.00	9.00	16.00	16.00	0.00	9.00	16.00	16.00	0.00	9.00
25	8.00	7.97	0.33	8.94	8.00	7.73	3.34	8.42	8.00	6.67	16.68	6.07
	10.00	9.97	0.27	8.95	10.00	9.73	2.67	8.53	10.00	8.67	13.34	6.66
	14.00	13.97	0.19	8.97	14.00	13.73	1.91	8.67	14.00	12.67	9.53	7.33
	16.00	15.97	0.17	8.97	16.00	15.73	1.67	8.71	16.00	14.67	8.34	7.54
50	8.00	7.93	0.91	8.87	8.00	7.27	9.14	7.71	8.00	4.35	45.68	0.98
	10.00	9.93	0.73	8.90	10.00	9.27	7.31	7.97	10.00	6.35	36.54	2.59
	14.00	13.93	0.52	8.93	14.00	13.27	5.22	8.26	14.00	10.35	26.10	4.42
	16.00	15.93	0.46	8.94	16.00	15.27	4.57	8.36	16.00	12.35	22.84	4.99
75	8.00	7.86	1.78	8.69	8.00	6.57	17.84	5.87	8.00	0.87	89.18	0.37
	10.00	9.86	1.43	8.75	10.00	8.57	14.27	6.50	10.00	2.87	71.34	0.53
	14.00	13.86	1.02	8.82	14.00	12.57	10.19	7.21	14.00	6.87	50.96	0.72
	16.00	15.86	0.89	8.84	16.00	14.57	8.92	7.44	16.00	8.87	44.59	1.18

Considering the evolution of the bridge service life and the different corrosion scenarios, it is possible to notice that the corrosion effects expressed in terms of steel reinforcement diameter variation are more evident for the steel rebars characterized by diameter equal to 8.00 and 10.00 mm representing the stirrups. Following what is indicated in previous Section 3, the corrosion effects due to the carbonation begin to develop after 13.5 years from the construction of the bridge. In fact, considering the age of structure ranging between 0 and 13.5 years, the corrosion effects are not yet present.

After 25 years since the construction of the viaducts, the corrosion effects become significant only considering the high corrosion level with $i_{corr} = 5$ μA/m² reaching values of steel reinforcement area reduction equal to 16.68 % considering an initial steel rebar diameter $d_0 = 8.00$ mm and 13.34 % for the steel rebars diameter $d_0 = 10.00$ mm. In the case of initial steel diameters equal to 14.00 mm and 16.00 the area reduction is characterized by values below 10.00 %.

Considering the case of 50 years after the construction time, the corrosion effects show significant values of steel reinforcement reduction area also in the case of moderate corrosion level ($i_{corr} = 1$ μA/m²) where, however, there are no reduction values that rich 10.00 % (9.14 % in the case of steel reinforcement characterized by initial diameter $d_0 = 8.00$ mm). Focusing attention on the high corrosion scenario ($i_{corr} = 5$ μA/m²) significant values of steel reinforcement area reduction have been obtained. In particular, considering the steel rebars characterized by $d_0 = 8.00$ mm the area reduction (ΔA_s) is equal to 45.68 % while for $d_0 = 10.00$ mm reaches 36.54 mm. Also, for steel reinforcement diameter equal to 14.00 mm and 16.00 mm, ΔA_s is greater than 20% (respectively 26.10 % and 22.84 %).

After 75 years from the viaducts construction also considering the moderate corrosion level ΔA_s is characterized by values greater than 10.00% for the steel reinforcement having diameter equal to 8.00 mm ($\Delta A_s = 17.84$ %), 10.00 mm ($\Delta A_s = 14.27$ %) and 14.00 mm ($\Delta A_s = 10.19$ %). Only for steel rebars diameter $d_0 = 16.00$ mm, ΔA_s shows a value less than 10%, equal to 8.92 %. In the case of high corrosion level very important steel reinforcement area reduction values are achieved: 89.19 % for $d_0 = 8.00$ mm, 71.34 % for $d_0 = 10.00$ mm, 50.96 % for $d_0 = 14.00$ mm and 44.59 % for $d_0 = 16.00$ mm.

Figs. 5, 6 and 7 report the variation of the moment-curvature diagram of the gross-section of the highest pier as a function of the age of the bridges. It is possible to notice that for all the analysed bridges, after 25 years of service life, the slight and moderate corrosion scenarios do not affect the

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load-bearing capacity and the ductility of the piers gross-section. This can be explained with the moderate steel reinforcement reduction, according to as reported in Table 7.

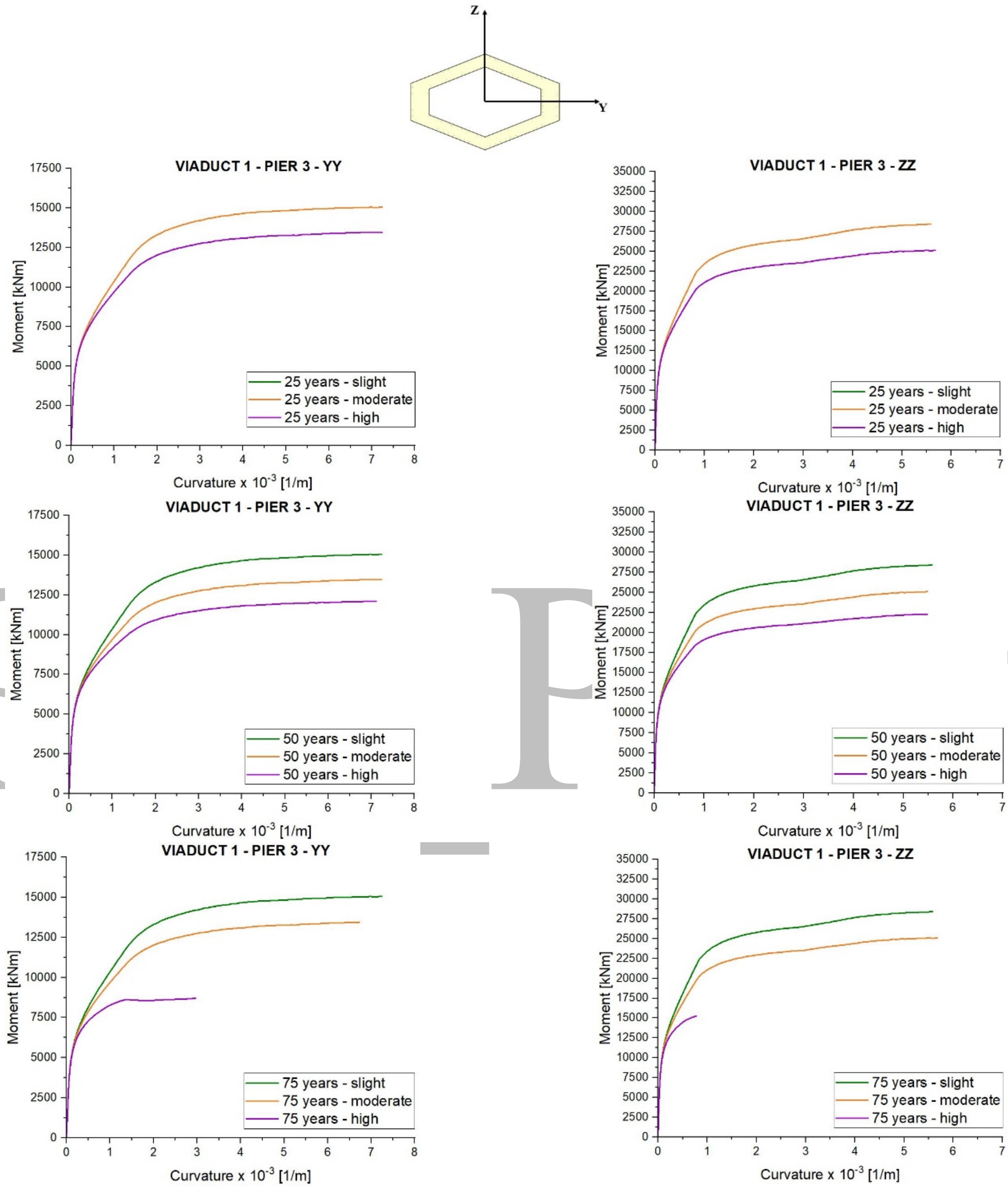

Figure 5. M-χ diagrams of the pier 3 of the viaduct 1 as a function of ageing (polygonal cross section)

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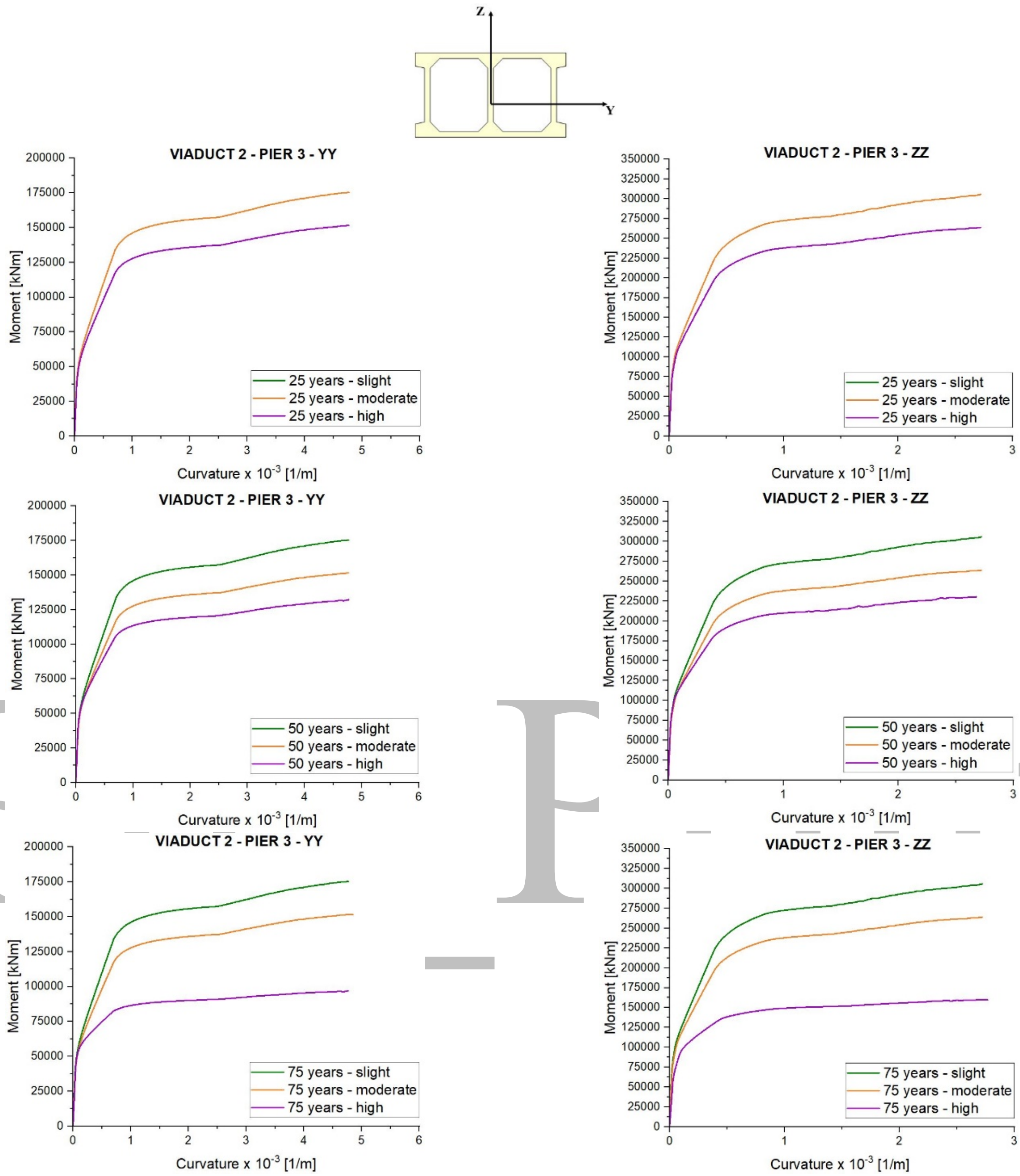

Figure 6. M-χ diagrams of the pier 3 of the viaduct 2 as a function of ageing (multi-channel cross section)

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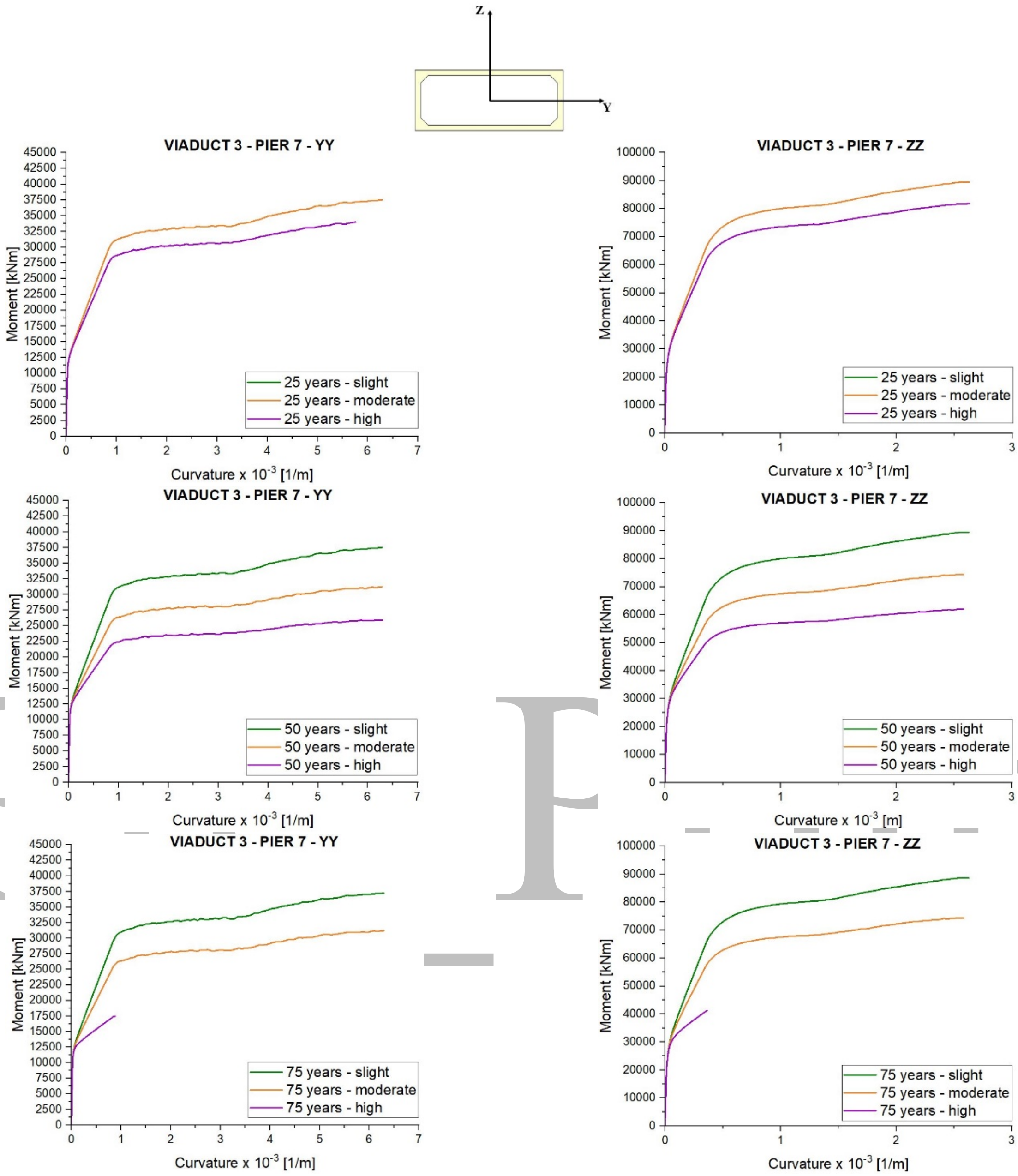

Figure 7. M-χ diagrams of the pier 7 of the viaduct 3 as a function of ageing (box cross section)

The evolution of the corrosion effects with the service life of the viaducts, leads to a progressive reduction of the stiffness of the piers which slightly influence the dynamic behaviour of the structures in terms of increment of the value of the first natural periods. Fig. 8 summarizes the modal analysis results obtained for the Viaduct 1.

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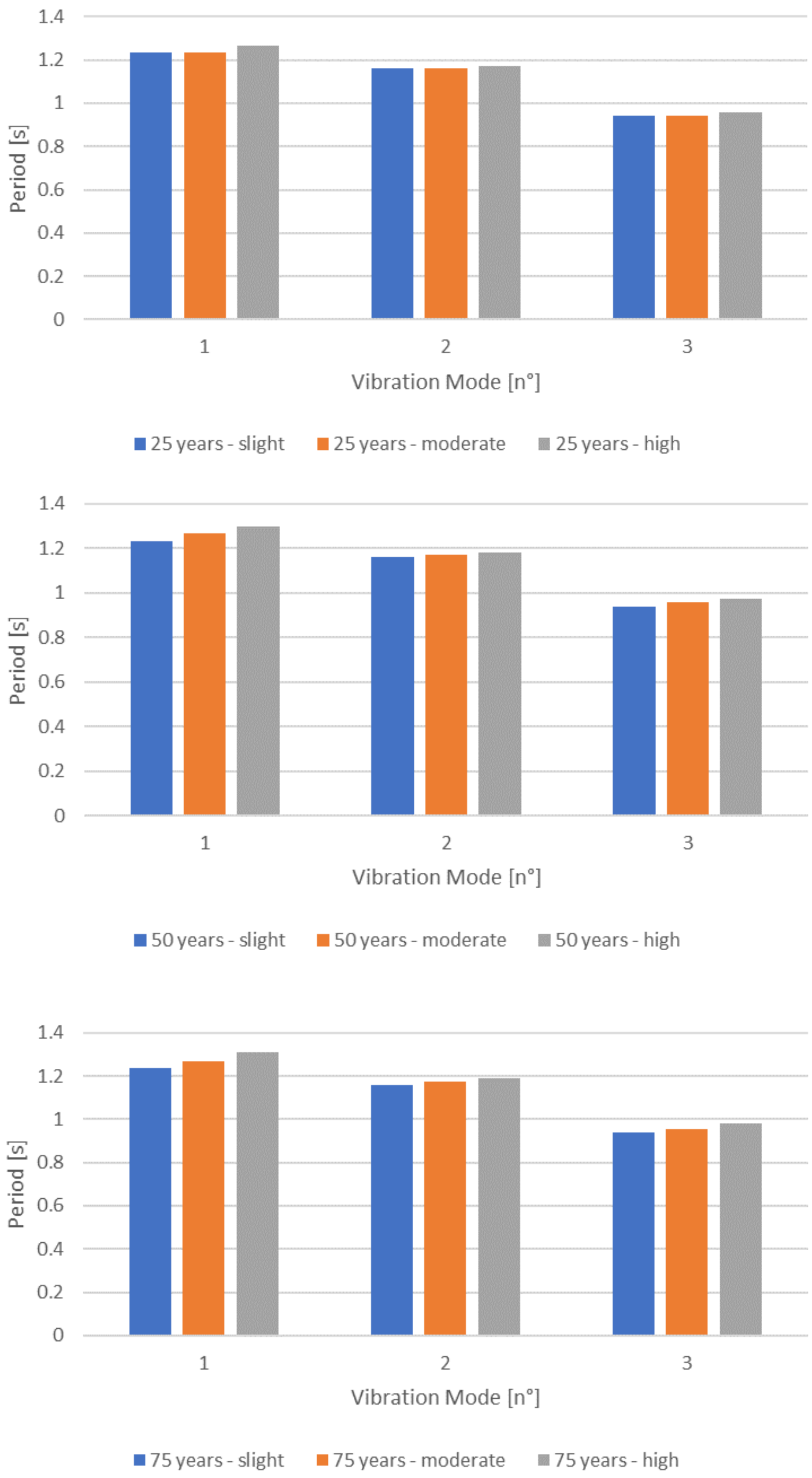

Figure 8. Fundamental periods of the Viaduct 1.

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As mentioned before, to evaluate the seismic vulnerability of the bridges at the construction time and after 13.5 years, 25 years, 50 years and 75 years, a series of time history analyses have been carried out according to [32] using seven seismic records for each viaduct listed in Table 5 and considering the seismic action acting on 0°, 45° and 90° from the longitudinal axis of the viaducts. The risk indices (RI) denote the ratio between the peak ground acceleration which leads to collapse of the first structural elements, considering both ductile and brittle failure mechanism, and the design peak ground acceleration reported in Table 2 and the ratio between the related return periods, as shown in the following Eqs. (5) and (6):

$$RI_{PGA} = \frac{PGA_C}{PGA_D} \tag{5}$$

$$RI_{RP} = \left(\frac{T_{R,C}}{T_{R,D}}\right)^{0.41} \tag{6}$$

Value of risk index less than one characterizes a viaduct with a significant risk to collapse under seismic action, while risk indices equal or greater than one defines a seismically safe structure.
Tables 8, 9 and 10 show the values of the risk indices obtained for the different corrosion levels during the age of the three viaducts.

Table 8. Risk indices for viaduct 1.

Viaduct 1	Corrosion level	25 years			50 years			75 years		
		0°	45°	90°	0°	45°	90°	0°	45°	90°
RI_{PGA}	Slight	1.571	1.986	1.457	1.571	1.986	1.457	1.571	1.986	1.457
	Moderate	1.571 (0.00%)	1.986 (0.00%)	1.457 (0.00%)	1.100 (-30.00%)	1.514 (-23.74%)	1.014 (-30.40%)	0.834 (-46.91%)	0.916 (-53.88%)	0.814 (-44.13%)
	High	1.100 (-30.00%)	1.514 (-23.74%)	1.014 (-30.40%)	0.914 (-41.82%)	1.243 (-37.41%)	0.878 (-39.74%)	0.714 (-54.55%)	0.500 (-74.82%)	0.371 (-74.54%)
RI_{RP}	Slight	1.755	2.391	1.588	1.755	2.391	1.588	1.755	2.391	1.588
	Moderate	1.755 (0.00%)	2.391 (0.00%)	1.588 (0.00%)	1.119 (-36.23%)	1.671 (-30.11%)	1.086 (-31.61%)	0.885 (-51.28%)	1.001 (-65.88%)	0.886 (-44.20%)
	High	1.119 (-36.23%)	1.671 (-30.11%)	1.086 (-31.61%)	0.925 (-47.29%)	1.242 (-48.06%)	0.912 (-42.57%)	0.636 (-63.76%)	0.507 (-78.79%)	0.429 (-72.98%)

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Table 9. Risk indices for viaduct 2.

Viaduct 2	Corrosion level	25 years			50 years			75 years		
		0°	45°	90°	0°	45°	90°	0°	45°	90°
RI_{PGA}	Slight	1.671	2.143	1.514	1.671	2.143	1.514	1.671	2.143	1.514
	Moderate	1.671 (0.00%)	2.143 (0.00%)	1.514 (0.00%)	1.314 (-21.37%)	0.757 (-64.67%)	0.543 (-64.15%)	0.911 (-45.48%)	0.537 (-74.94%)	0.422 (-72.13%)
	High	1.314 (-21.37%)	0.757 (-64.67%)	0.543 (-64.15%)	1.229 (-26.45%)	0.614 (-71.35%)	0.443 (-70.74%)	0.500 (-70.07%)	0.400 (-81.33%)	0.300 (-80.15%)
RI_{RP}	Slight	1.957	2.746	1.711	1.957	2.746	1.711	1.957	2.746	1.711
	Moderate	1.957 (0.00%)	2.746 (0.00%)	1.711 (0.00%)	1.410 (-27.94%)	0.751 (-72.65%)	0.534 (-68.77%)	0.987 (-49.56%)	0.584 (-78.73%)	0.478 (-72.06%)
	High	1.412 (-27.94%)	0.751 (-72.65%)	0.534 (-68.77%)	1.291 (-34.03%)	0.617 (-77.53%)	0.426 (-75.10%)	0.488 (-75.06%)	0.379 (-86.20%)	0.279 (-83.69%)

Table 10. Risk indices for viaduct 3.

Viaduct 3	Corrosion level	25 years			50 years			75 years		
		0°	45°	90°	0°	45°	90°	0°	45°	90°
RI_{PGA}	Slight	2.300	1.986	1.300	2.300	1.986	1.300	2.300	1.986	1.300
	Moderate	2.300 (0.00%)	1.986 (0.00%)	1.300 (0.00%)	2.000 (-13.04%)	1.714 (-13.67%)	1.129 (-13.19%)	1.236 (-46.26%)	1.344 (-32.33%)	0.933 (-28.23%)
	High	2.000 (-13.04%)	1.714 (-13.67%)	1.129 (-13.19%)	1.214 (-43.35%)	1.343 (-32.38%)	0.971 (-25.31%)	0.857 (-62.74%)	0.900 (-54.68%)	0.743 (-42.84%)
RI_{RP}	Slight	2.364	2.026	1.299	2.364	2.026	1.299	2.364	2.026	1.299
	Moderate	2.364 (0.00%)	2.026 (0.00%)	1.299 (0.00%)	2.041 (-13.67%)	1.735 (-14.33%)	1.128 (-13.17%)	1.295 (-45.21%)	1.412 (-30.30%)	0.921 (-29.10%)
	High	2.041 (-13.67%)	1.735 (-14.33%)	1.128 (-13.17%)	1.214 (-48.64%)	1.342 (-33.76%)	0.913 (-29.71%)	0.875 (-62.99%)	0.913 (-54.94%)	0.750 (-42.26%)

The obtained results show a significant reduction of the risk indices as the age of the viaducts increases, considering the moderate and the high corrosion levels. The slight corrosion scenario, instead, does not significantly influence the seismic performance of the existing RC viaducts, according to the low levels of the steel reinforcement reduction area reported in Table 7. After 75 years from the construction of the viaducts the risk indices, both in terms of peak ground acceleration (PGA) and in terms of related return period (RP), are characterized by reduction values greater than 30% considering the seismic input acting on 0°, 45° and 90° from the longitudinal axis of the considered bridge for all the three corrosion scenarios. The maximum reduction is observed for the Viaduct 2 considering the seismic signal acting at 45° from the longitudinal axis of the bridge where it is equal to 81.33 % for the risk index related to PGA and 86.20 % for the risk index related to RP. It is important to highlight that considering the high corrosion scenario, all the risk indices obtained have a value smaller than one, which characterized structures with a significant risk to collapse if subject to a seismic event equal to the design one, defined according to [37]. On the contrary, for moderate corrosion level, Viaduct 3 maintains value of risk indices greater than one for each direction of the seismic action.

After 50 years from the construction of the viaducts, only Viaduct 2 shows risk indices values less than one in the case of moderate corrosion scenario (with reduction that exceed 60%), when considering a seismic signal action on 45° and 90° from the longitudinal axis of the bridge while Viaducts 1 and 3 are characterized by values of risk indices always close or greater than one also for the high corrosion level.

Considering the case of 25 years from the construction of the viaducts, a significant reduction of the seismic performance is obtained only in the case of high corrosion scenario (i_{corr} = 5 μA/m^2) which reaches reduction values of the risk indices even greater than 60 % for the Viaduct 2. Viaduct 1 is characterized by reduction of the risk indices close to 30 % in the case of high corrosion level while Viaduct 3 shows much more limited reduction values equal to about 13 %.

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Figs. 9, 10 and 11 report the variation of the risk indices normalized to the initial value (calculated at the construction time of the viaducts) as a function of the age of the viaducts.

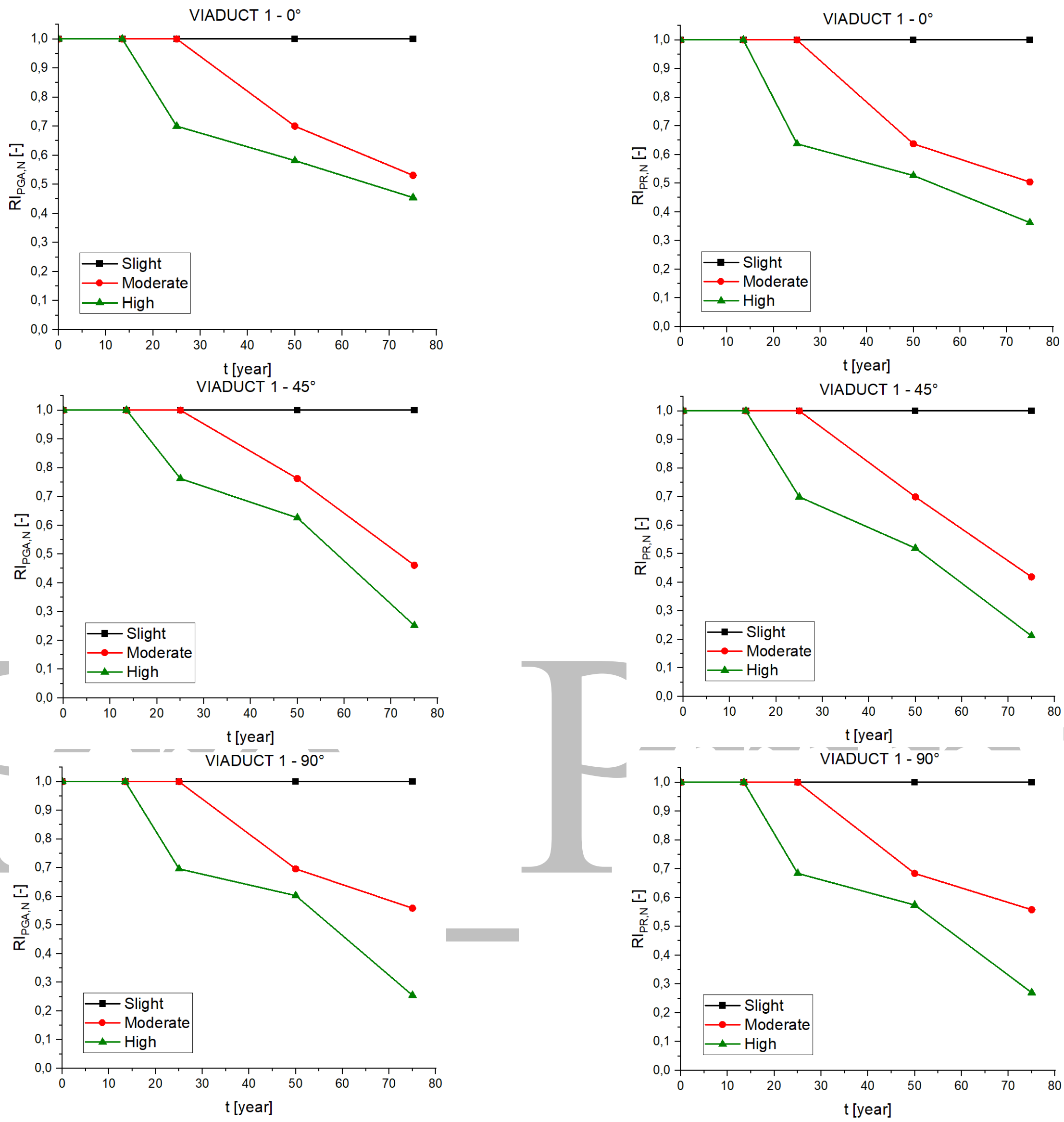

Figure 9. Trend of risk indices obtained for the Viaduct 1.

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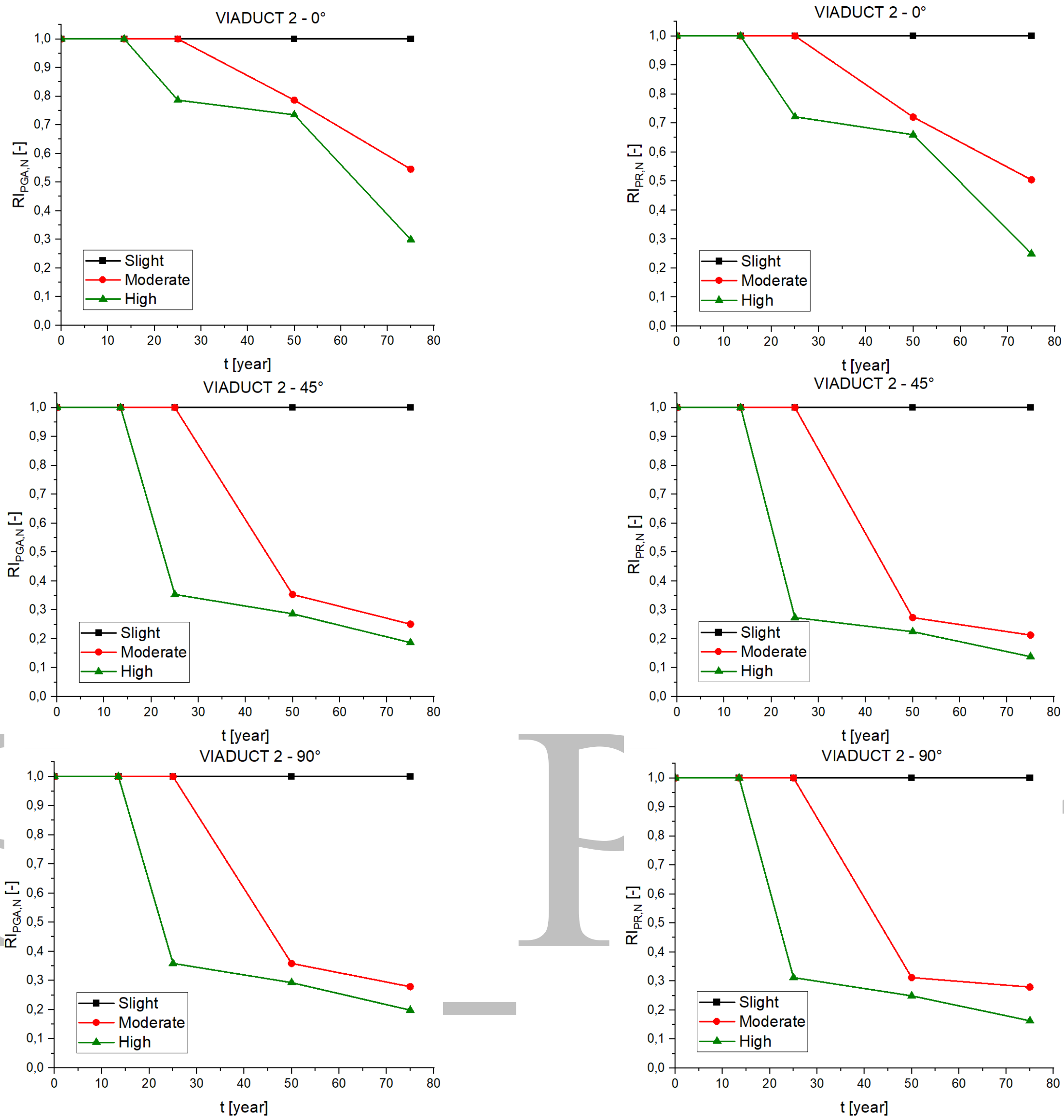

Figure 10. Trend of risk indices obtained for the Viaduct 2.

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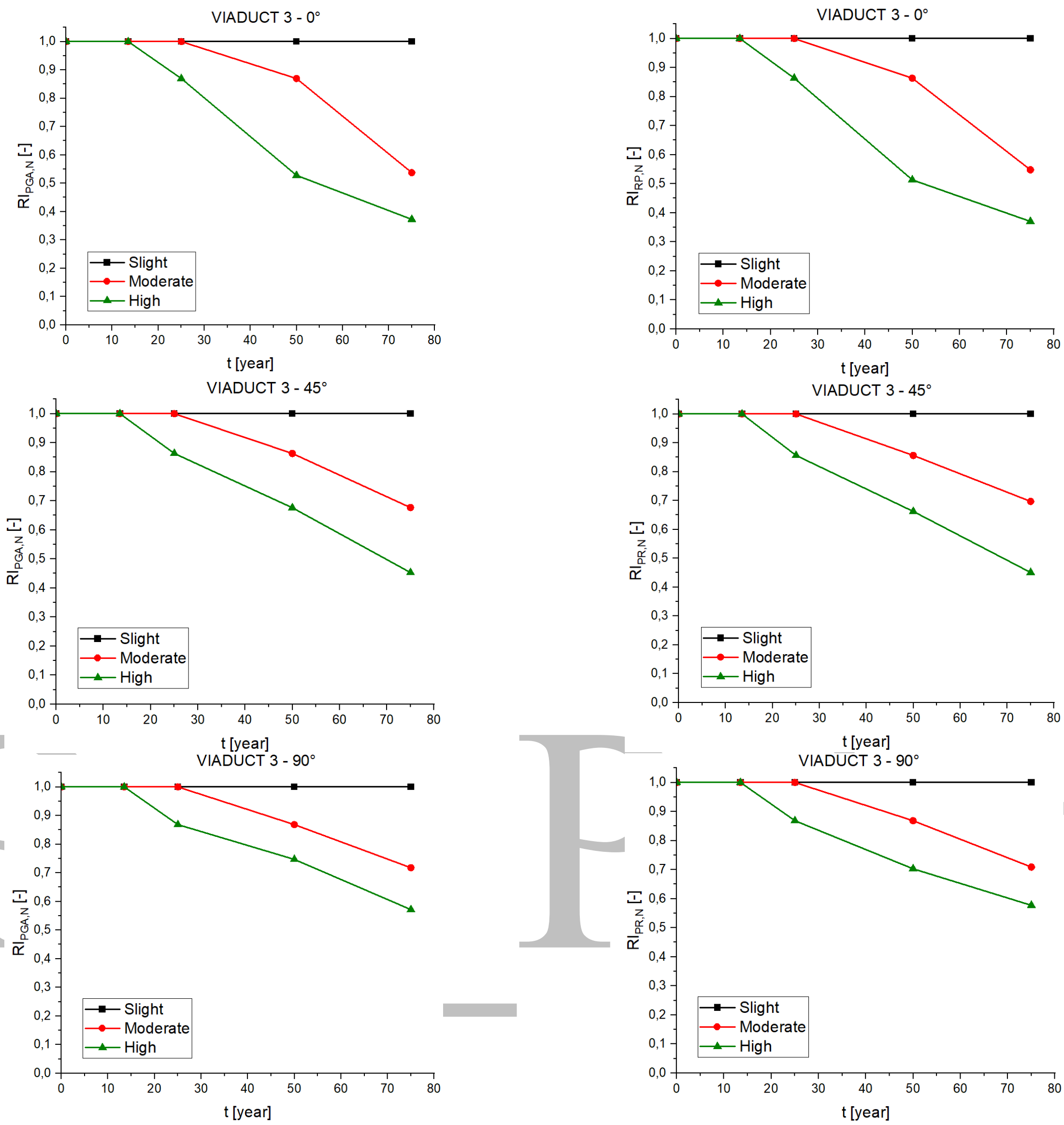

Figure 11. Trend of risk indices obtained for the Viaduct 3.

Starting from the obtained risk indices, it is possible to calculate the intervention time (IT), related to the considered limit state (life-safety limit state in this work), which characterized the viaducts, through the following Eq.(7) [52]:

$$IT = 0.105 \cdot \frac{\min(RP)}{C_u} \tag{7}$$

where IT is the intervention time (in years), min(RP) is the minimum return period obtained starting from the risk indices calculated and C_u is the coefficient for use category taken, in this work, equal to 2 according to as reported in [37]. Table 11 listed the evolution of the intervention times obtained for the three case studies as a function of the age of the viaducts.

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Table 11. Intervention times.

	Viaduct 1			Viaduct 2			Viaduct 3		
	25	50	75	25	50	75	25	50	75
IT [years]	60.93	39.79	6.33	10.78	6.22	2.21	66.84	39.90	24.70

It is possible to notice that the intervention time decreases significantly with increasing aging of the viaducts, considering a high corrosion level for which the values of the lower risk indices were obtained. In fact, considering as reported in Table7, the increase in the level of corrosion related to the age of the bridge leads to significant reduction values in terms of steel reinforcement areas, especially for the stirrups characterized by a smaller diameter than the longitudinal steel reinforcement. As a consequence, the shear strength of the piers decreases which regulate the activation of the brittle collapse mechanism (Tables 8, 9 and 10). This fact is more evident for viaduct 2 which is characterized by stirrups with smaller diameter (8 mm) than viaduct 1 and 3 (10 mm).

5. Conclusions

In this paper the correlation between the seismic performance of existing RC viaducts and the evolution of the corrosion effects due to the carbonation with the age of the structures (0-75 years) has been analysed. Corrosion effects are considered only in terms of steel reinforcement area reduction by means of analytical formulation. Three different corrosion levels are analysed (slight, moderate and high) characterized by a different value of corrosion current density (i_{corr}).

The seismic performance of the viaducts has been evaluated considering simplified finite element model implemented using Timoshenko beam elements and plastic hinges located at the base of the piers. A series of non-linear time history analyses (NTHA) have been carried out on three existing RC viaducts realized between 1960's and 1970's and located in moderate/high seismicity areas of Northern Italy, in order to obtain the values of the risk indices, express in terms of ratio between the peak ground acceleration which leads to collapse of the first monitored structural elements considering both ductile and brittle failure mechanism and the design peak ground acceleration and the ratio between the related return periods, useful to calculate the intervention time which characterizes the analysed structure.

The corrosion effects due to the carbonation start after 13.5 years from the construction of the structure and for this reason, three different time steps are considered in this work to evaluate the relation between the seismic performance of the existing RC motorway viaducts and the corrosion effects: 25, 50 and 75 years after the construction of the structure. From the results obtained, it is possible to notice that:

- the slight corrosion scenario, characterized by a value of $i_{corr} = 0.1$ μA/mm^2, does not influence the seismic performance of the analysed viaducts even after 75 years from the construction of the structure;
- considering the case of 25 years from the construction of the RC viaducts, significant variations of the seismic performance of the analysed viaducts have been obtained only considering the high corrosion scenario ($i_{corr} = 5$ μA/mm^2). The moderate corrosion scenario does not influence the bearing-capacity of the viaducts under seismic actions;
- after 50 years, considering the moderate corrosion level ($i_{corr} = 1$ μA/mm^2) only Viaduct 2 is characterized by risk indices values less than one, when the seismic signal acts on 45° and 90° from the longitudinal axis of the bridge. On the contrary Viaducts 1 and 3 show values of risk indices always close or greater than one also considering the high corrosion scenario;

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- analysing the results obtained after 75 years from the construction time of the viaducts, a significant reduction of the seismic performance is observed also in the case of moderate corrosion level for each case study.

These results obtained are useful to schedule the correct maintenance interventions in such a way as to give priority to intervention within the same motorway network, also considering the decrease of the intervention time as a function of the age of the structures.

Finally, further development of the work is to consider the possible occurrence of the buckling phenomenon of the longitudinal reinforcements as the effect of the corrosion increase, as reported in [50,51]

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Please cite this document as: M. Zucca, P. Crespi, F. Stochino, M. L. Puppio, M. Coni,
Maintenance interventions period of existing RC motorway viaducts located in moderate/high seismicity zones, *Structures*, 2023, 47: 976-990. https://doi.org/10.1016/j.istruc.2022.11.135

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Please cite this document as: M. Zucca, P. Crespi, F. Stochino, M. L. Puppio, M. Coni, Maintenance interventions period of existing RC motorway viaducts located in moderate/high seismicity zones, *Structures*, 2023, 47: 976-990. https://doi.org/10.1016/j.istruc.2022.11.135