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# EVALUATION OF MAINTENANCES INTERVENTION PERIOD ON STOCK OF EXISTING RC BRIDGES SUBJECT TO DAMAGE PHENOMENA

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## Abstract

*During last decades, the attention of the scientific community on the evaluation of the load-bearing capacity of existing reinforce concrete (RC) bridges has significantly increased even as a result of the recent collapses which have occurred. Particular attention is focused on the seismic performance of the existing RC bridges especially if located in areas characterized by high seismicity level such as Italy. One of the most important problems that affects the seismic performance of this structures is the presence of corrosion effects due to carbonation phenomenon which may involve the steel reinforcements of the piers.*
*In this paper the evaluation of the seismic vulnerability of an existing RC bridge located in Italy and built around the 1960's is presented, considering three different corrosion scenarios (slightly, moderate, and high). Non-linear time history analysis has been performed considering a simplified Finite Element Model (FEM) where the structural elements have been implemented with beam elements and the non-linear behavior of the piers has been introduced using appropriate concentrated plastic hinges. The carbonation effects have been modelled considering the piers steel reinforcement area reduction as a function of the age of the bridge. Risk indices evaluated as the ratio of the peak ground acceleration leading to collapse of the first structural element and the design peak ground acceleration, are calculated to define the seismic vulnerability of the analyzed structure.*

**Keywords:** Existing RC Bridges, Seismic Vulnerability, Time-History Analysis, Corrosion Effects.

## 1 INTRODUCTION

Correct scheduling of maintenance interventions of existing reinforced concrete (RC) bridges represents one of the most important aspects useful to guarantee an adequate safety level of these strategic infrastructures. Focusing attention on seismic performance of existing RC bridges, corrosion effects due to carbonation acting on the RC piers can lead to a significant reduction of their load-bearing capacity when subjected to horizontal loads [1-4].

Several Italian RC motorway bridges were built between 1960's and 1970's and, consequently require a series of maintenance interventions in order to maintain an adequate safety level [5]. Furthermore, these structures have been realized without considering the presence of the seismic action, according to the design codes of the time.

To evaluate the seismic behavior of existing RC bridges, several approaches based on non-linear analysis method have been proposed during last decades. One of the most used methods is the pushover analysis but it is applicable only in presence of structures characterized by a dynamic behavior with a predominant translational vibration mode [6]. For this reason, the use of multi-modal pushover approach has been extended by [7,8] to the evaluation of the seismic vulnerability of existing RC bridges. Different probabilistic approaches, based on the use of fragility curves which define the seismic response of these structures have been proposed in literature [9-11].

Another approach developed in recent years and used for the evaluation of the seismic performance of existing RC bridges is the Incremental Dynamic Analysis (IDA) [12,13]. A simple alternative of the IDA method is represented by IMPAB approach proposed by [14] 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. A good compromise between computational effort and accuracy of the results 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.

Different approaches have been developed to consider the corrosion effects due to carbonation phenomena on the seismic performance of existing RC bridges. In this paper, a simplified analytical method which considers the corrosion effects only in terms of steel reinforcements area reduction has been taken into account. The approach has been applied to an RC existing motorway bridge located in Northern Italy, considering three different corrosion scenarios (slight, moderate and high) and evaluating the evolution of the seismic performance of the bridge until 75 years from the construction time, through the execution of a series of NTHA analyses. The seismic vulnerability of the bridge has been expressed in terms of appropriate risk indices based on the ratio between the peak ground acceleration which leads to the collapse of the first monitored structural element and the design peak ground acceleration obtained from the Italian Design Code [15] and the ratio of the related return periods. These indices have been useful to calculate the minimum time intervention period through the application of a simplified relation suggested by [16].

## 2 STRUCTURAL MODELLING

To evaluate the seismic behavior of existing RC bridges, the simplified procedure described in [17] have been used. The approach is based on the implementation of simplified 3D Finite Element models (FEM) where the main structural elements which characterize the bridge as the piers, the pier caps and the deck have been modelled with beam elements while the elastomeric bearings have been introduced in the FEM using elastic links with translational and rotational stiffnesses have been calculated according to [18]. The connection between the elastomeric bearings and the other structural elements is realized through a series of rigid links as shown in Figure 1.

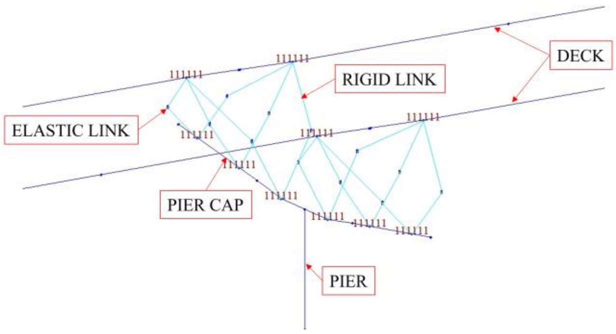


Figure 1: Elastomeric bearings connection.

The pier foundations have been considered as perfect constrain applied to the node at the base of each pier while the presence of the abutments have been represented by perfect constrains applied to the node located at the base of each elastic links representing the elastomeric bearings located at the abutment-deck interface. In order to reproduce the correct dynamic behavior of the bridge, the reduction of the bending stiffness of the gross-section of each pier due to the concrete cracking has been considered through the application of appropriate scale factors calculated starting from the moment-curvature diagram (M-$\chi$) which characterize the gross-section of each pier, according to as reported in [19]. On the contrary, the deck stiffness is not reduced despite the formation of bridge decks cracks it is still in the elastic phase during seismic events [20]. In the FEM, the contribution of structural and non-structural masses is considered while the presence of traffic load is according to [15].

Two failure mechanisms of the piers have been monitored: (i) the ductile collapse mechanism related to the moment-curvature diagram of the gross-section and which is characterize by a linear elastic portion followed by a hardening branch and (ii) the brittle collapse mechanism regulated by 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.

To take into account the non-linear behavior of the materials, Kent and Park model [21] (Figure 2a) and Park Strain Hardening [22] (Figure 2b) constitutive law have been adopted respectively for the concrete and for the steel reinforcement.

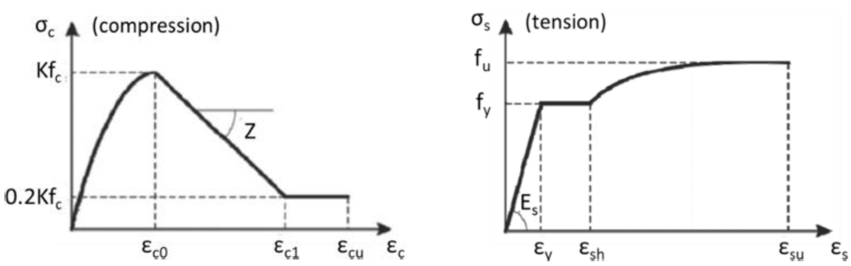


Figure 2: (a) Kent and Park and (b) Park Strain Hardening model.

Appropriate plastic hinges, calculated according to [23,24] have been applied to the base of each pier where the formation of the ductile mechanism is expected. The considered ultimate limit states are the following: (i) for the ductile failure mechanism, the achievement of ¾ of the ultimate rotation $\vartheta_u$ (Figure 3a) while (ii) for the brittle failure mechanism the overcome of the shear resistance $V_R$ of the considered structural element (Figure 3b) calculated considering the formulation proposed in [25] for the cyclic shear resistance based on the sum of three different terms depending on the axial load, the concrete strength, and the stirrups.

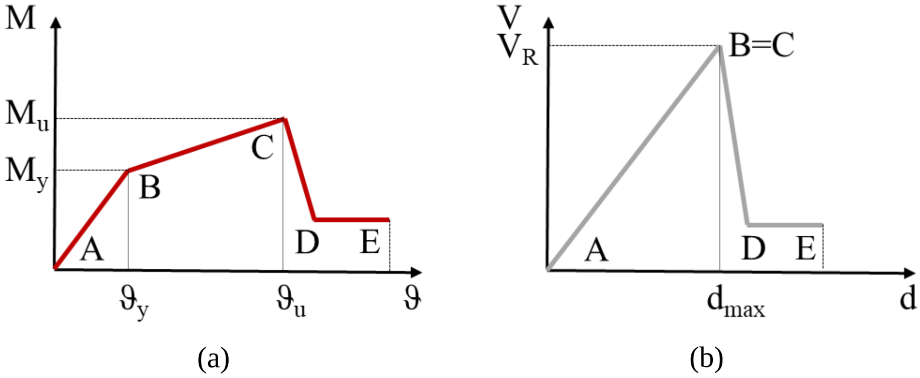


Figure 3: (a) Ductile and (b) Brittle collapse mechanism.

The corrosion effects have been introduced in the FEM considering a simplified analytical approach based on the progressive reduction of steel reinforcement diameter [26] and where the penetration law in a generic concrete volume is characterized by a parabolic behavior (Equation 1):

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

where $s$ is the thickness of the carbonated layer, $t$ the time and $k$ the penetration rate coefficient. The parameter $n$, which depends on the concrete characteristics, can be taken equal to 2 considering that the existing RC bridge analyze was built between the 1960's and 1970's with normal compacted concrete [27]. It possible to calculate the residual service life ($t_{res}$) of the bridge starting from the initiation time ($t_i$), the propagation time ($t_p$) and the maximum expected rebar radius reduction ($P_{lim}$) following the Equation 2 reported in [28]:

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

where $t$ is the bridge age, $k$ 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 Equation 2 is not useful. In this work the estimation of the correlation between the reduction of the steel reinforcement diameter and the seismic performance of the bridges has been considered, through the following Equations 3 and 4:

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

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

where $d$ is the reduction of the steel reinforcement diameter, $d_0$ is the initial steel reinforcement diameter and $P(t)$ is the corroded thickness.

Three different corrosion scenarios have been analysed: slight, moderate and high corrosion level characterized by a value of $i_{corr}$, respectively, equal to 0.1 μA/cm$^2$, 1 μA/cm$^2$ and 5 μA/cm$^2$. The reduction of steel reinforcement diameter is obtained ad the difference between the initial diameter ($d_0$) and the diameter referred to the time of interest ($d(t)$). It is possible to notice that the reduction of the steel reinforcement diameter strictly depends on the initiation time ($t_i$) considered as statistical variable and the corrosion current density ($i_{corr}$) calculated considering as reported in the design codes or obtained from experimental results. Considering an initial value of cover thickness equal to 25 mm, the iterative process has been developed for different concrete type. The following values of other parameters has been considering in this work: w/c = 0.6, $t_i$ = 13.5 years [28] while concrete compressive strength $f_{ck}$ = 28 MPa, penetration rate coefficient k = 0.0116 and steel rebar ultimate deformation $\varepsilon_{u,0}$ = 9 % have been considered as constants for each analyzed corrosion scenario.

To evaluate the seismic performance of the existing RC bridges subjected to corrosion phenomena due to carbonation, a series of non-linear time history analyses (NTHA) have been performed considering both ductile and brittle collapse mechanism in the same FEM in order to take into account the interaction between the two failure mechanisms.

## 3 CASE STUDY

The approach described in the previous Section has been applied to an existing RC bridge located in Northern Italy. Figure 4 shows the FE model of the considered bridge. The main seismic characteristics of the site where the bridge was built are the following: Soil type = C (evaluated according to Eurocode 8) and PGA = 0.156 g.

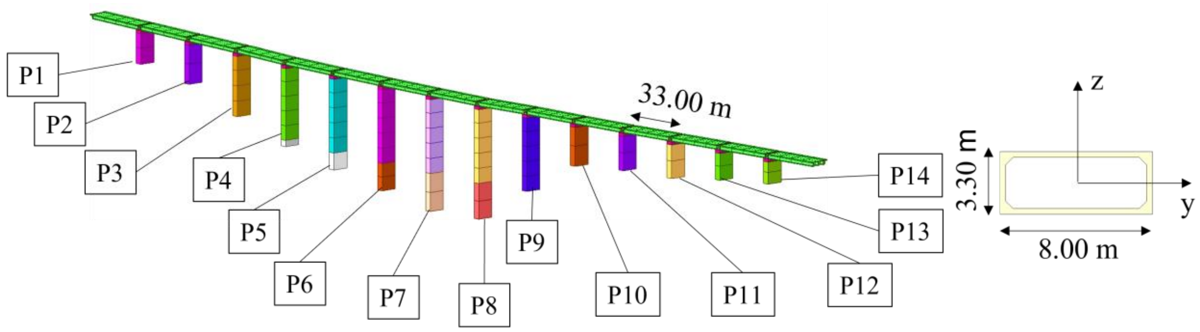


Figure 4: FEM.

In particular, the bridge 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 analyzed, have been made with $f_{ck}$ = 28 MPa concrete and $f_{yk}$ = 440 MPa steel. The main characteristics of the bridge and of the piers are listed, respectively, in Table 1 and Table 2.

| Spans [n°] | Length [m] | Elastomeric bearings [n°] | Piers [n°] | Piers shape [-] | Piers thickness [m] |
|---|---|---|---|---|---|
| 15 | 515 | 2 x 3 | 14 | Rectangular hollow | 0.35 |

Table 1: Bridge structural characteristics.

| Pier [n°] | Cross section dimensions [m] | Height [m] | Longitudinal steel reinforcement [-] | Transverse steel reinforcement [-] |
|---|---|---|---|---|
| 1 | 8.0 x 3.3 | 14.50 | 164Ø14 | Ø10/20 |
| 2 | 8.0 x 3.3 | 18.50 | 164Ø14 | Ø10/20 |
| 3 | 8.0 x 3.3 | 28.53 | 164Ø14 | Ø10/20 |
| 4 | 8.0 x 3.3 | 37.00 | 164Ø14 | Ø10/20 |
| 5 | 8.0 x 3.3 | 43.00 | 164Ø14 | Ø10/20 |
| 6 | 8.0 x 3.3 | 48.00 | 164Ø14 | Ø10/20 |
| 7 | 8.0 x 3.3 | 53.00 | 164Ø14 | Ø10/20 |
| 8 | 8.0 x 3.3 | 51.97 | 164Ø14 | Ø10/20 |
| 9 | 8.0 x 3.3 | 34.76 | 164Ø14 | Ø10/20 |
| 10 | 8.0 x 3.3 | 18.46 | 164Ø14 | Ø10/20 |
| 11 | 8.0 x 3.3 | 16.31 | 164Ø14 | Ø10/20 |
| 12 | 8.0 x 3.3 | 15.98 | 164Ø14 | Ø10/20 |
| 13 | 8.0 x 3.3 | 12.75 | 164Ø14 | Ø10/20 |
| 14 | 8.0 x 3.3 | 10.46 | 164Ø14 | Ø10/20 |

Table 2: Piers main characteristics.

Considering the seismic parameters of the site where the bridge was built, seven spectrum-compatible accelerograms have been obtained using Rexel software [29]. Table 3 reports the fundamental characteristics of the seismic signals derived from the European Strong-Motion Database (ESD).

| Event [-] | Station ID [-] | Year [-] | PGA [m/s$^2$] | PGV [m/s] | Magnitude $M_w$ [-] |
|---|---|---|---|---|---|
| Umbria Marche | ST223 | 1997 | 0.567 | 0.048 | 5.3 |
| Lazio Abruzzo | ST152 | 1984 | 1.444 | 0.112 | 5.9 |
| Ionian | ST8 | 1973 | 2.498 | 0.255 | 5.8 |
| Umbria Marche | ST232 | 1997 | 0.501 | 0.012 | 5.3 |
| Basso Tirreno | ST47 | 1978 | 1.493 | 0.083 | 6.0 |
| Umbria Marche | ST223 | 1978 | 0.326 | 0.031 | 5.3 |
| Izmit | ST3273 | 1999 | 1.387 | 0.089 | 5.8 |

Table 3: Seismic signals considered in this work.

The first three fundamental natural periods which characterize the dynamic behavior of the bridge are: $T_1$ = 2.64 s, $T_2$ = 1.87 s and $T_3$ = 1.75 s.

Table 4 indicates the reduction of the steel reinforcement diameter and area due to corrosion effects evaluated for the three above-mentioned different corrosion levels taking into account different time intervals since the construction of the bridge.

| t | Slight Corrosion Scenario $i_{corr}$ = 0.1 [μA/cm$^2$] | | | | Moderate Corrosion Scenario $i_{corr}$ = 1 [μA/cm$^2$] | | | | High Corrosion Scenario $i_{corr}$ = 5 [μA/cm$^2$] | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $d_0$ | d | $\Delta A_s$ | $\varepsilon_u$ | $d_0$ | d | $\Delta A_s$ | $\varepsilon_u$ | $d_0$ | d | $\Delta A_s$ | $\varepsilon_u$ |
| [year] | [mm] | [mm] | [%] | [%] | [mm] | [mm] | [%] | [%] | [mm] | [mm] | [%] | [%] |
| 0-13.5 | 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 |
| 25 | 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 |
| 50 | 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 |
| 75 | 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 |

Table 4: Area and diameter reduction of the steel reinforcement.

Taking into account the bridge service life and the considered three corrosion levels, it is possible to highlight that the corrosion effects are more evident for the transverse steel reinforcement having diameter equal to 10.00 mm. Considering as reported in the previous Section 2, the corrosion effects 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 bridge, the corrosion effects become significant only considering the high corrosion level with $i_{corr}$ = 5 μA/m$^2$ reaching values of steel reinforcement area reduction equal to 13.34 % for the steel rebars diameter $d_0$ = 10.00 mm.

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/m$^2$) where, however, there are no reduction values that rich 10.00 %. Focusing attention on the high corrosion scenario ($i_{corr}$ = 5 μA/m$^2$) significant values of steel reinforcement area reduction has been obtained. In particular, considering the steel rebars characterized by $d_0$ = 10.00 mm the area reduction ($\Delta A_s$) is equal to 36.54 %.

After 75 years from the bridge construction also considering the moderate corrosion level $\Delta A_s$ is characterized by values greater than 10.00% for the steel reinforcement having diameter equal 10.00 mm ($\Delta A_s$ = 14.27 %) and 14.00 mm ($\Delta A_s$ = 10.19 %). In the case of high corrosion level very important steel reinforcement area reduction values are achieved: 71.34 % for $d_0$ = 10.00 mm and 50.96 % for $d_0$ = 14.00 mm.

Figure 5 reports the trend of the moment-curvature diagram of the gross-section of the pier 7 as a function of the age of the bridges. It is possible to highlight that after 25 years of service life, the slight and moderate corrosion scenarios do not affect the load-bearing capacity and the ductility of the pier gross-section.

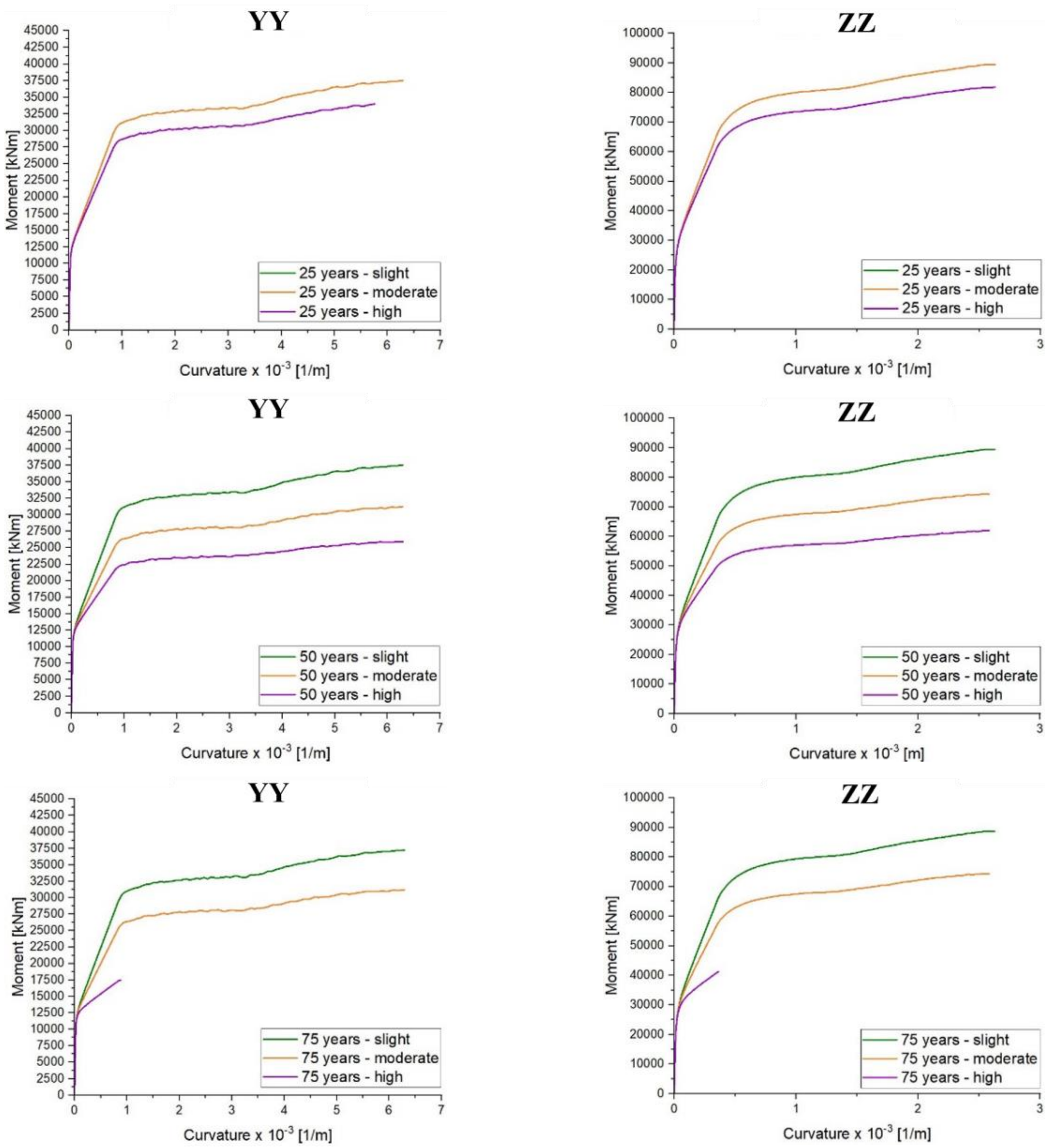


Figure 5: Moment-curvature diagrams of the pier 7.

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 behavior of the structures in terms of increment of the value of the first natural periods.

In order to define the seismic performance of the analyzed existing RC bridge at the construction time and after 13.5 years, 50 years and 75 years, non-linear time-history analyses have been performed using the seven seismic signals listed in Table 3 considering the seismic load action on 0°, 45° and 90° from the bridge longitudinal axis. Two different risk indices, reported in following Equations 5 and 6, have been considered for the evaluation of the seismic performance of the bridge:

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

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

where $PGA_C$ is the value of the peak ground acceleration which leads to the collapse of the first monitored structural element, $PGA_D$ is the design value of the peak ground acceleration evaluated considering as reported in [15] and $T_{R,C}$ and $T_{R,D}$ are, respectively, the related return periods. Risk index characterized by a value less than one define a bridge with a significant risk to collapse under seismic load, while risk index equal or greater than one defines a seismically safe structure.

Table 5 summarizes the value of the risk indices obtained considering the different corrosion scenarios and the age of the bridge.

| | Corrosion level | 25 years 0° | 25 years 45° | 25 years 90° | 50 years 0° | 50 years 45° | 50 years 90° | 75 years 0° | 75 years 45° | 75 years 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%) |

Table 5: Risk indices.

It is possible to notice an important reduction of the value of the risk indices as a function of the age of the bridge, mainly considering the moderate and the high corrosion scenarios. The slight corrosion scenario does not influence the seismic behavior of the bridge, according to the low levels of the steel reinforcement reduction area shown in previous Table 4. After 75 years from the construction of the bridge 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 moderate and the high corrosion scenarios. It is possible to notice that taking into account the high corrosion scenario, all the risk indices obtained both for ductile and brittle collapse mechanism, are characterized by a value smaller than one, that define a structure with a significant risk to collapse if subject to a seismic event equal to the design one. On the contrary, for moderate corrosion scenario, the bridge maintains value of risk indices greater than one for each direction of the seismic action.

After 50 years from the construction, the bridge is characterized by values of risk indices always close or greater than one for all the corrosion scenarios analyzed.

Considering the case of 25 years from the construction, the bridge shows much more limited reduction values of the risk indices, equal to about 13 %.

Figure 6 summarizes the trend of the above-mentioned risk indices normalized to the initial value (evaluated when the bridge was built) as a function of the age of the bridge.

Starting from the values of the risk indices calculated for the different corrosion scenarios, it is possible to obtain the intervention time (IT), related to the considered limit state (life-safety limit state in this work), which characterized the bridge, considering the Equation 7 [16]:

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

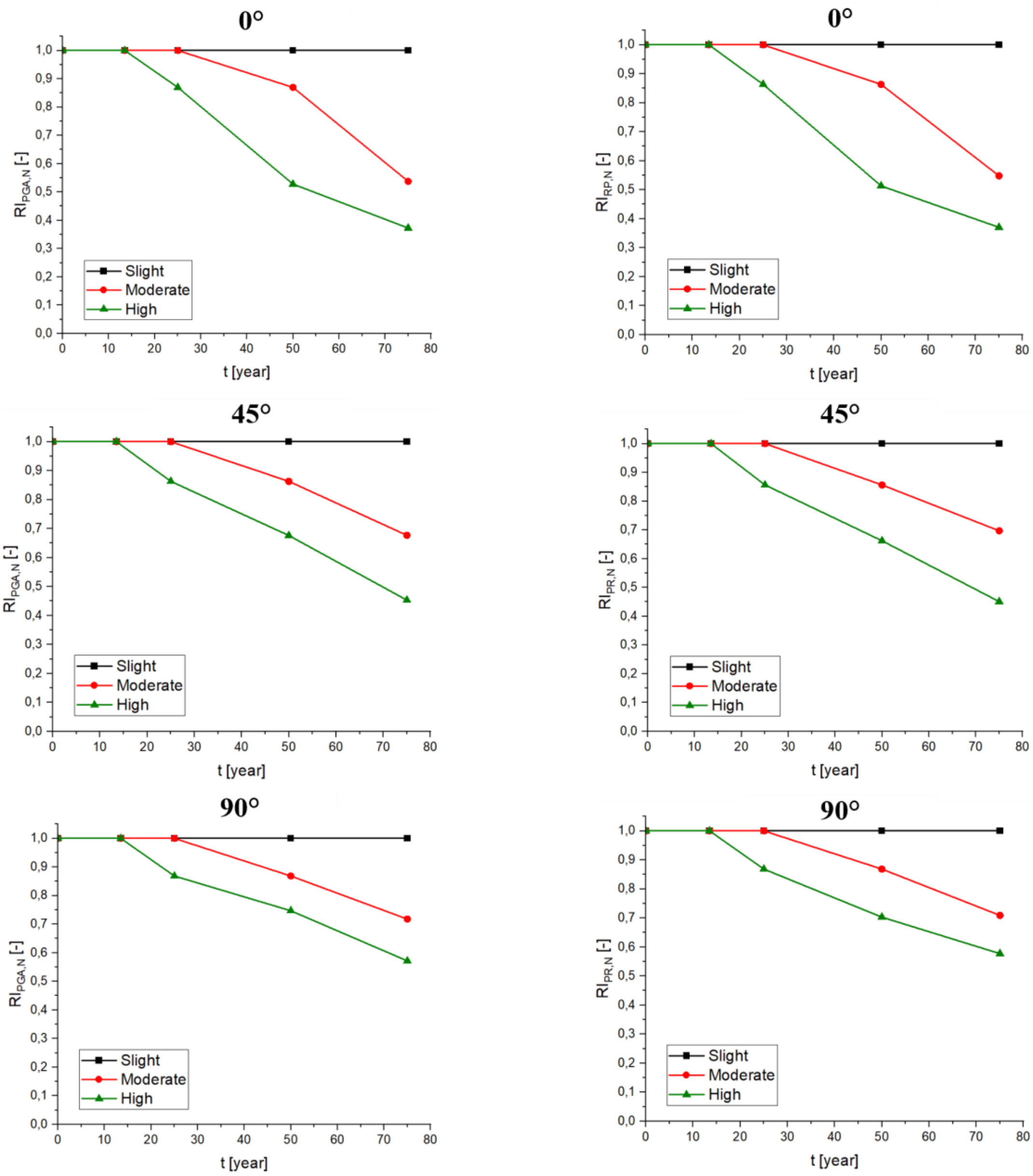


Figure 6: Evolution of the normalized risk indices value as a function of the bridge age.

where IT indicates the intervention time (in years), min(RP) represents 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 [15]. Table 6 summarizes the evolution of the intervention time as a function of the age of the analyzed bridge.

| | 25 | 50 | 75 |
|---|---|---|---|
| IT [years] | 66.84 | 39.90 | 24.70 |

Table 6: Evolution of the intervention time (IT).

It is possible to highlight 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, 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 yielding to the activation of the brittle collapse mechanism.

## 4 CONCLUSIONS

In this work the influence of the corrosion effects due to carbonation on the seismic performance of existing RC bridges considering the age of the structure (0-75 years) is analyzed. In particular, the corrosion effects are considered only in terms of steel reinforcement area reduction by means of analytical formulation. Three different corrosion scenarios are evaluated: (i) slight, (ii) moderate and (iii) high characterized by a different value of corrosion current density ($i_{corr}$).

The seismic behavior of the bridge has been defined through the implementation of a simplified finite element model using Timoshenko beam elements and plastic hinges located at the base of the piers. Different non-linear time history analyses (NTHA) have been performed on an existing RC bridge built between 1960’s and 1970’s to obtain the values of the risk indices, expressed in terms of the 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 analyzed 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 brdige 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 $i_{corr} = 0.1\ \mu A/mm^2$, does not influence the seismic performance of the bridge even after 75 years from the construction;
- considering the case of 25 years from the construction of the bridge, significant variations of the seismic performance have been obtained only considering the high corrosion scenario ($i_{corr} = 5\ \mu A/mm_2$). The moderate corrosion scenario does not influence the bearing-capacity of bridge under seismic actions;
- after 50 years, considering the moderate corrosion level ($i_{corr} = 1\ \mu A/mm^2$), the bridge shows values of risk indices always close or greater than one also considering the high corrosion scenario;
- analyzing the results obtained after 75 years from the construction time, a significant reduction of the seismic performance is observed also in the case of moderate corrosion level.

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

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