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Seismic reliability analysis of a r.c. arch bridge

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Abstract

Seismic reliability analysis (SRA) is a powerful tool able to combine seismic hazard and structural fragility for computing the probability of failure of a specific damage state of interest. In this work, a SRA has been applied to a specific case study of an open spandrel RC arch bridge located in North-eastern Italy, adopting as suitable engineering demand parameters for both the ductile and fragile failure mechanism of the columns between the RC arches and the RC beams grillage. Collapse seismic fragilities are computed for each structural element and, finally, the bridge seismic reliability is computed in a system reliability-based format.

Keywords: Seismic risk map, insurance, Italy, risk mitigation, earthquake, engineering, cost-benefit analysis.

1. Introduction

Nowadays, an ever-increasing attention is dedicated to a reliability assessment of structures, with specific reference to a set of performance targets. Engineers have thus to guarantee adequate safety levels for the entire service-life of the structure of interest. When dealing with seismic reliability analysis, the main contributions to consider are the likelihood of each ground shaking level at the construction site, and the most probable structural response due to a

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specific shaking value. The goal of the seismic reliability analysis is thus to couple these two aspects and find suitable indicators for representing the structural safety level (Zanini et al. 2019; Zanini and Hofer 2019).

In this context, one of the most adopted risk indicators is the so-called failure rate λ_f that represents the annual rate of exceeding a specific damage state for the structure. This indicator is usually computed assuming that the occurrence of the main earthquakes at the construction site can be represented by a Homogenous Poisson Process (HPP) and that there is not damage accumulation on the structure. Under these hypotheses, the structural failure itself is an HPP, whose unique parameter λ_f can thus be used for computing the failure probability in any time interval. For a specific damage state, λ_f can be computed as

$$\lambda_f = \int_{im} P[f|im]|d\lambda_{im}| \tag{1}$$

where λ_{im} is the so-called hazard curve and represents the seismicity at the construction site. In Eq. (1) $P[f|im]$ represents structural vulnerability, i.e. the probability of reach and exceed a specific damage level conditioned on a given value of ground motion intensity measure im. In many cases, im is assumed to be the peak ground acceleration (PGA), i.e. the spectral acceleration associated to a structural period equal to zero, but any spectral acceleration $S_a(T)$ can be adopted. Commonly, the computation of λ_{im} is based on the Probabilistic Seismic Hazard Analysis (PSHA, Cornell 1968, McGuire 1995), which associates to each $IM = im$ value the corresponding annual rate of events exceeding im at the construction site. Once computed λ_{im}, $|d\lambda_{im}|$ in Eq. (1) can be obtained by deriving the hazard curve

$$|d\lambda\_im| = -(d\lambda\_im)/dim \tag{2}$$

Regarding $P[f|im]$, different approaches have been proposed in literature for the calibration of this function and are all based on results carried out with a set of non-linear dynamic analysis. Among all procedures, the most adopted are the Incremental Dynamic Analysis (IDA, Vamvatsikos and Cornell 2004), the so-called Cloud Analysis (Jalayer and Cornell 2003), and the Multi-Stripes Analysis (MSA, Baker 2015). Finally, in order to allow a direct comparison with target structural safety values provided in the current technical codes for construction, from λ_f it is possible to derive the failure probability in the time window of T years as

$$P_{f,T} = 1 - e^{-\lambda_f \cdot T} \tag{3}$$

and thus, the associated reliability index

$$\beta_T = -\Phi^{-1}(P_{f,T}) \tag{4}$$

Finally, β_T has to be compared with a target value of seismic reliability β_{target}, for guaranteeing a suitable safety margin

$$\beta_T \geq \beta_{target} \tag{5}$$

2. Case study

This section describes a complete seismic reliability analysis performed for an existing double-span open-spandrel reinforced concrete RC arch bridge located in the Vicenza district (lat. 46.01, lon. 11.63), northeastern Italy and built in 1946. Figures 1 (Figures 1 and 2). The entire procedure can be subdivided in three main steps. The first one step consists in the seismic hazard computation for the construction site, while in the second dynamic non-linear structural analyses are performed for deriving the structural fragility. Finally, hazard and fragility are combined for assessing the seismic reliability of the bridge.

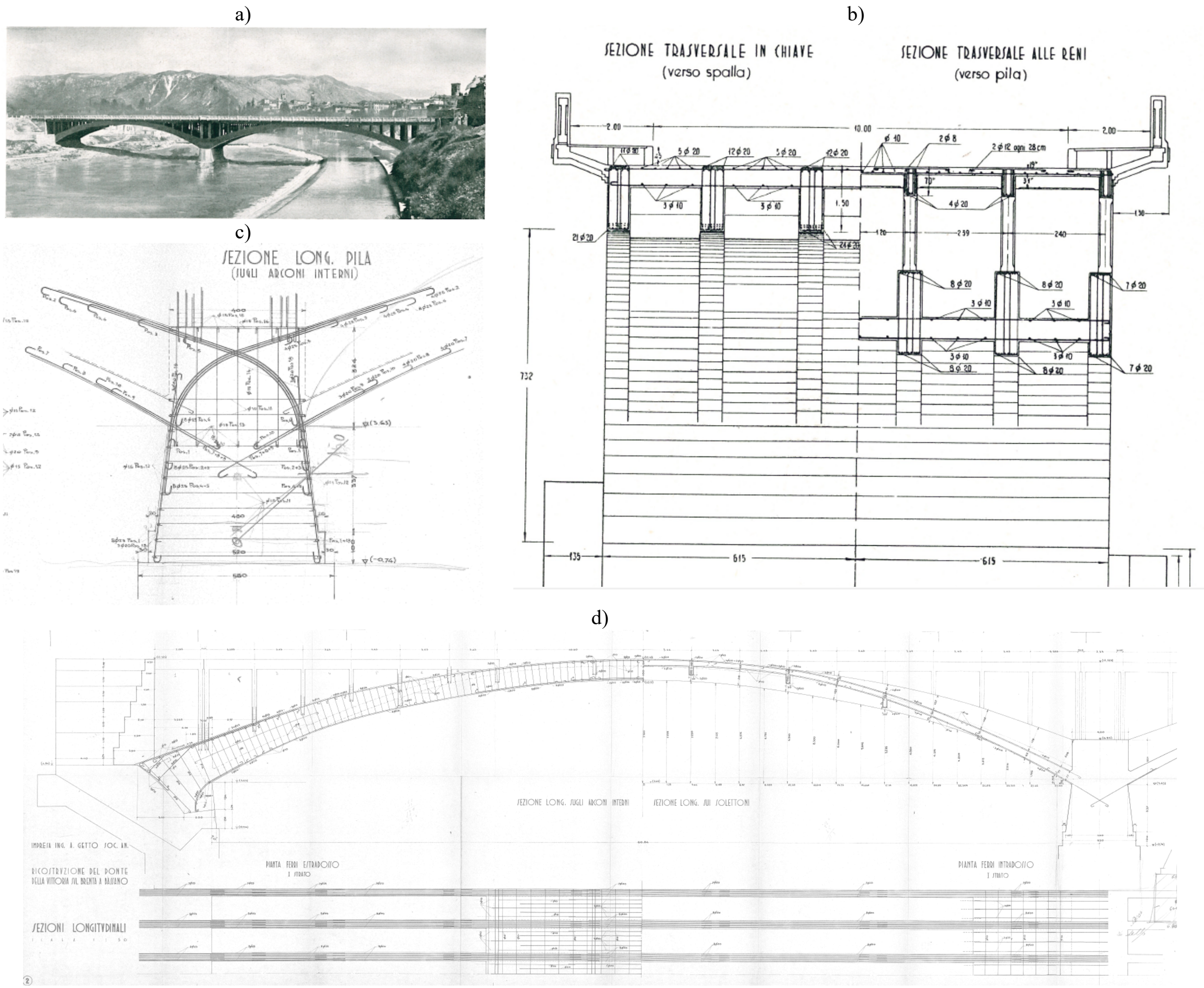

Fig. 1. Two-span RC arch bridge case study (a) and original drawings with reinforcement detailing of the deck (b), pier (c) and arches (d) (from Ufficio del Genio Civile di Vicenza 1948).

To compute λ_{im}, the seismic hazard map for Italy, provided by the National Institute of Geology and Volcanology (INGV), was used. The map is based on a 5-km span grid and, for each node, seismic hazard data are provided, with reference to nine return times, that correspond to exceedance probabilities of 2, 5, 10, 22, 30, 39, 50, 63 and 81% in 50 years, respectively. To compute the failure rate a continuous hazard function is needed. Since INGV provides hazard data (values of the 16th, 50th and 84th percentile) only for nine return times, median values were fitted with a quadratic function in the logarithmic space as:

$$\lambda(s) = k_0\, e^{(-k_1 \ln(s) - k_2 ln^2(s))} \tag{6}$$

In assessing seismic reliability, instead of the median hazard curve, it is more suitable to refer to the mean one which is possible to derive with the following equation:

$$\bar{\lambda}(s) = \lambda(s)\, e^{\left(\frac{1}{2}\beta_H^2\right)} \tag{7}$$

where β_H can be estimated as:

$$\beta_H = \frac{\ln(S_{84\%}) - \ln(S_{16\%})}{2} \tag{8}$$

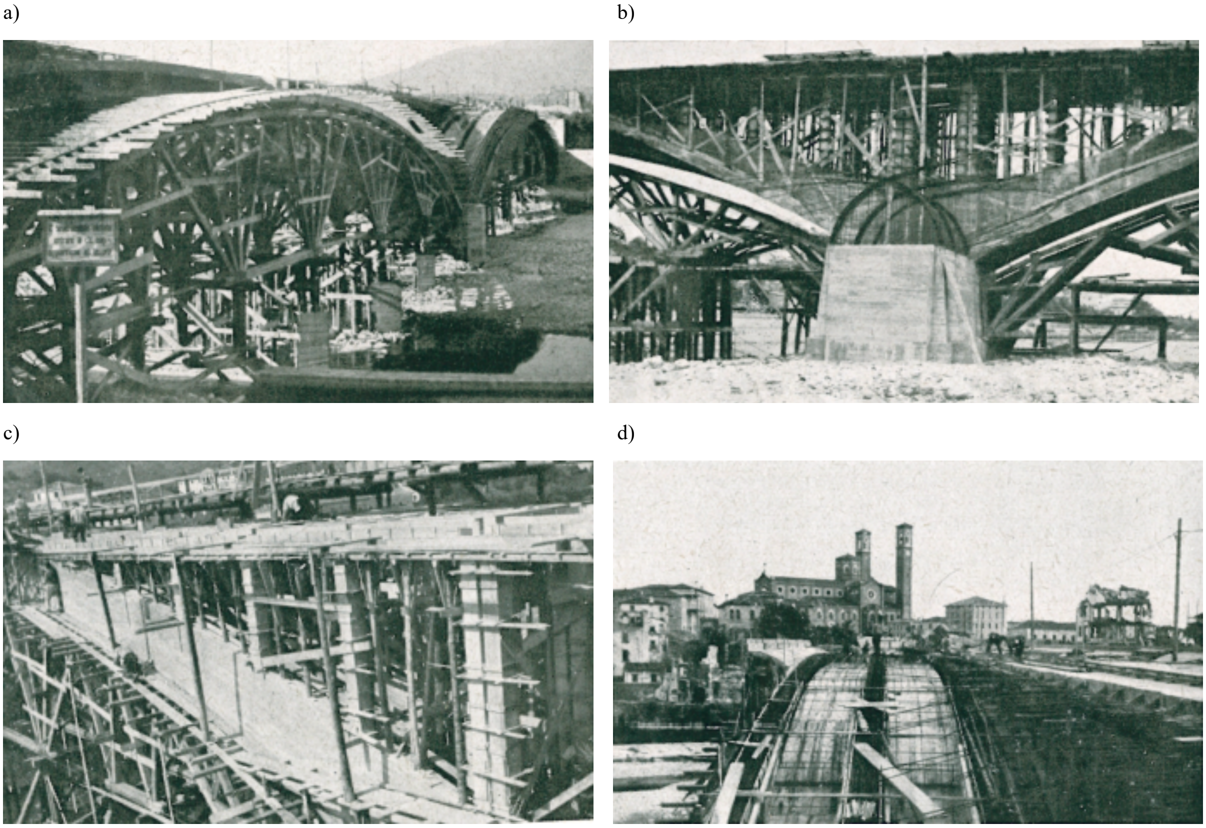

Fig. 2. Some pictures of the construction stages of the bridge in 1946 (from Ufficio del Genio Civile di Vicenza 1948).

The considered bridge is a double-span open-spandrel reinforced concrete RC arch bridge. It has an overall length or nearly 120 m (60 m per span) and each span is supported by 6 arches connected by a RC slab. The nonlinearities are concentrated on the arches and columns and are modelled through distributed plasticity, while the horizontal upper structure, formed by a beam grillage and slab, is considered elastic. The well consolidated Mander et al. (1988) model was used to model the concrete non-linear behaviour while the reinforcement was modelled through the Menegotto and Pinto (1973) model. In addition, three expansion joints are present in the deck structure, at the starting and final point and one over the middle pier with a small eccentricity with respect to the centreline and are modelled through gap elements. Because of limited available information, the foundational boundaries are considered as fully constrained. Figure 3 shows the 3D Finite Element model set up for the analysis purposes.

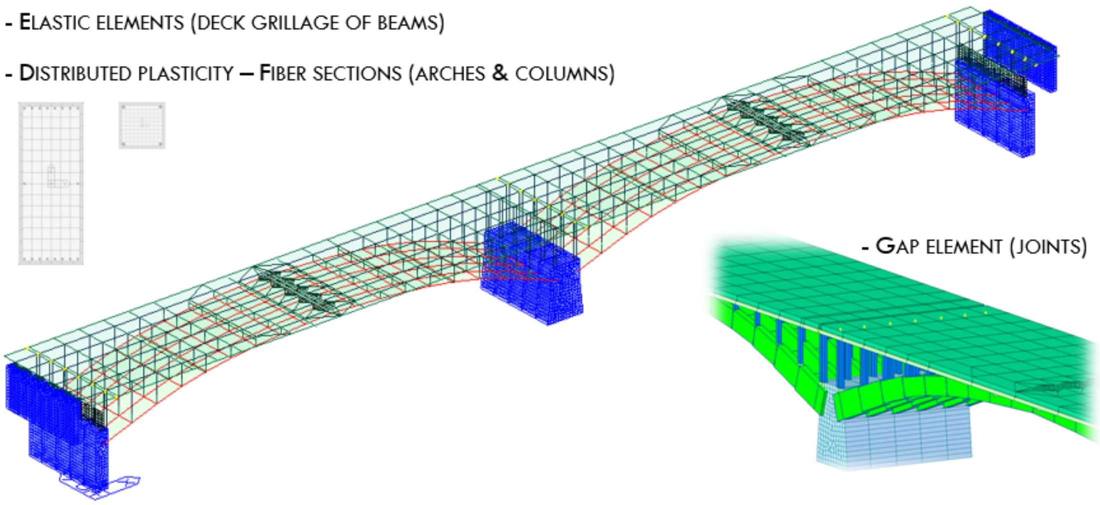

Fig. 3. 3D Finite Element Model and relevant modelling details.

Earthquake record selection is an important step in a seismic reliability analysis since it can easily affect the final results (Zanini et al. 2017). In the present paper a set of 35 ground motions, selected from the Engineering Strong Motion (ESM, Luzi et al. 2016) database, is used to perform non-linear time history analysis (NLTHAs) on the fem model of the selected bridge. Both horizontal components and the vertical one of the seismic waves are considered in the analysis. The main characteristics of the earthquake records are shown in Table 1. and Figure 4. The considered set covers Peak Ground Acceleration (PGA) values from 0 to nearly 1g, with magnitude varying between 4 and 7.5 while the epicentral distance (R) varies in most cases from 0 to 20 km with a maximum up to 60 km.

Table 1. Selected 3-D earthquake records.

ID #	Event	Date [d/m/y]	ID #	Event	Date [d/m/y]	ID #	Event	Date [d/m/y]
1	L'Aquila	06/04/2009	13	Ancona	14/06/1972	25	Azores Islands	09/07/1998
2	L'Aquila	06/04/2009	14	Central_Italy	26/10/2016	26	Greece	07/09/1999
3	Emilia	29/05/2012	15	Turkey	13/03/1992	27	Central Italy	26/10/2016
4	L'Aquila	06/04/2009	16	Turkey	01/05/2003	28	Albania	13/06/1993
5	Friuli	17/06/1976	17	Western Caucasus	03/05/1991	29	Central Italy	26/01/2003
6	Friuli	11/09/1976	18	Pyrgos	26/03/1993	30	Southern Greece	25/10/1984
7	Southern Italy	16/01/1981	19	Southern Greece	15/09/1986	31	Friuli	11/09/1976
8	Umbria-Marche	14/10/1997	20	Greece	08/11/2014	32	Norcia	19/09/1979
9	Northern Italy	07/06/1980	21	Greece	24/04/1988	33	Friuli	06/05/1976
10	Southern Italy	09/09/1998	22	Austria	06/05/1998	34	Ancona	14/06/1972
11	Ancona	21/06/1972	23	Greece	19/05/1995	35	Gibraltar	04/01/1994
12	Duzce	12/11/1999	24	Greece	14/07/1993	-	-	-

The fragility function represents the probability to reach and exceed a given damage state level, conditioned on a specific ground motion intensity measure IM=*im*, and its calibration is commonly based on results carried out with a set of NLTHAs. In the present paper, the Cloud Analysis (Vamvatsikos and Cornell 2004) approach is used. In this case, the investigated structure is subject to a limited set of n unscaled ground motions records and the fragility curve takes origin from the sample of n ground motions intensities and the corresponding sample of structural responses assuming the following form:

$$P[f|im] = P\left[EDP > \overline{edp}|im\right] = 1 - P\left[EDP \leq \overline{edp}|im\right] = 1 - \Phi\left[\frac{ln(\overline{edp}) - ln(edp)}{\beta}\right] \tag{9}$$

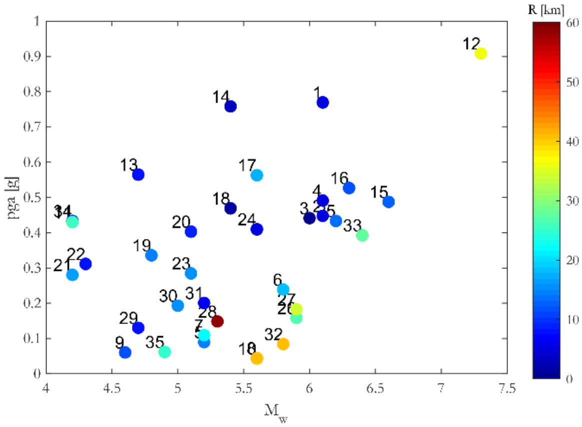

Fig. 4. Magnitude, epicentral distance and horizontal peak ground acceleration of the selected 3-D records.

In Eq. (9), $\overline{edp}$ is the median threshold value of the assumed structural limit state, and edp represents the median estimate of the demand that can be computed with a ln-linear regression model, as:

$$ln(edp) = a + b \cdot ln(im) \tag{10}$$

Finally, β is the standard deviation of the demand conditioned on im and can be estimated from the regression of the seismic demands as:

$$\beta = \sqrt{\frac{\sum_{i=i}^{n}\left[ln(edp_i) - \left(a + b \cdot ln(im_i)\right)\right]^2}{n-2}} \tag{11}$$

Therefore, to compute the fragility curves the Intensity Measure (IM) parameter and the Engineering Demand Parameter (EDP) have to be defined. In the present study, the PGA is the considered IM parameter while fragility curves were computed for both ductile and fragile failure mechanisms considering curvature ductility and shear strength as the respective EDPs. The curvature ductility μ_φ is defined as the ratio between the maximum curvature of a cross-section φ_i and the curvature corresponding to the steel yielding in the cross-section φ_y computed through a cross-section analysis. On the other hand, shear strength was defined as by the Italian Code for Construction (NTC 2018). For the ductile mechanism four Damage States (DS) i=1, 2, 3 and 4, respectively Slight, Moderate, Extensive and Complete while only the collapse (Complete) damage state can be considered for fragile failure. The respective curvature ductility threshold values were set as $\mu_{\varphi,DSi}$ = 1, 2, 4 and 7. Finally, system fragility curves were computed considering ductile and fragile as in series mechanisms. Fragility curves and reliability indexes were computed for both longitudinal and transversal directions for each column element and the lowest reliability index was chosen for the final considerations.

3. Results

Generally, in a standard framework, seismic reliability analysis is carried out considering the overall performance of the structure. In this study, the seismic reliability is assessed at an element scale. Retrofitting an existing bridge is not only an expensive operation but also consequences as traffic closure or limitation can cause great inconvenience and economic losses to daily users. When dealing with large scale structures, especially bridges, evaluating the performance level of the single elements can be of crucial importance for the decision-making process that comes after an assessment. In the present paper, the time-history response of each RC column was evaluated in terms of curvature demand for the ductile failure mechanism and in terms of shear strength for the fragile failure mechanism. Fragility curves were computed for each element for both horizontal directions (Figure 5), subsequently, the probability of failure P_f and reliability index β were computed following the procedure shown in section 1, considering a time interval of 1 year.

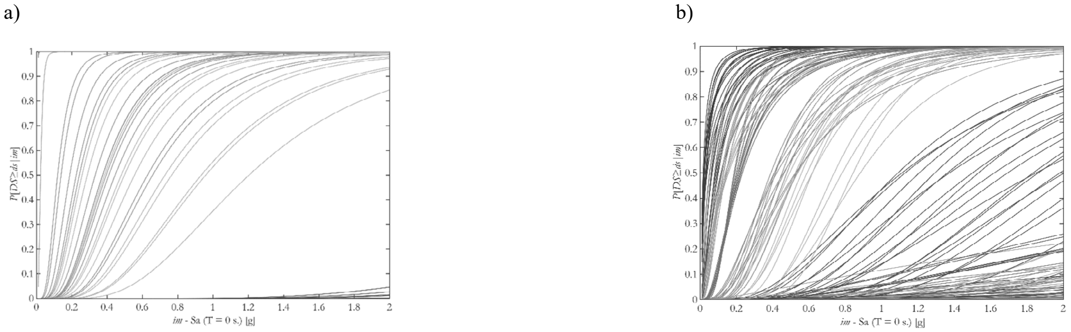

Fig. 5. Collapse fragility curves for each of the 168 RC columns: longitudinal (a) and transversal (b) directions.

In Figure 6 a), b) and c) the seismic reliability indexes for ductile and fragile failure mechanism and the resulting system from considering ductile and fragile as in series mechanisms are reported. It is immediately clear that the prevailing mechanism is the fragile one while the ductile one is less vulnerable. Also, short columns near the centerline of the RC arches are the most vulnerable elements of the bridge, especially for the fragile mechanism due to their stocky shape.

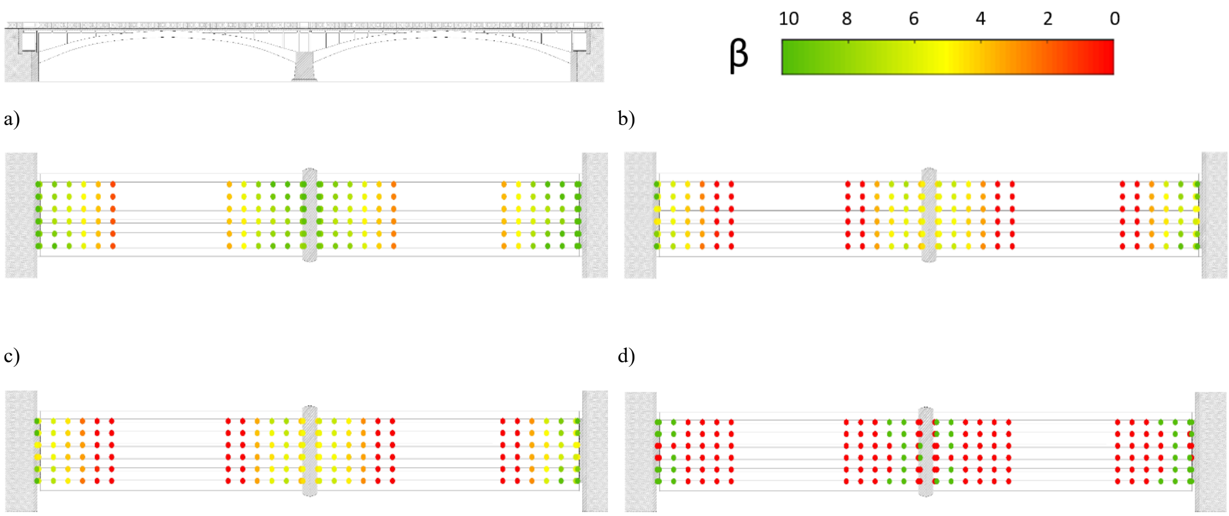

Fig. 6. Seismic reliability assessment: ductile (a) and fragile (b) failure mechanisms, system reliability (c) and seismic reliability check (d).

To check the collapse safety level of the elements, reliability index β was compared to the target reliability index provided by the Eurocode 0 (European Committee for Standardization 2015) and set at the value of 4.7 for a time window of 1 year. The results are shown in Figure 6d, in red unsafe elements with reliability indexes lower than the target one, and in green elements evaluated as safe, with reliability indexes higher than 4.7. The seismic reliability assessment highlights how only 28.5% of the RC column element can be checked as safe in reliability terms, while stocky elements near the center of the span are subjected to a higher seismic risk with respect to the other elements.

4. Conclusions

A seismic reliability analysis of an existing double-span open spandrel RC arch bridge was herein detailed with the aim to probabilistically estimate its seismic structural safety. The present work applied such methodology to the specific case study, adopting as suitable engineering demand parameter the curvature ductiliy of the columns between the RC arches and the RC beams grillage, quantifying collapse seismic fragilities for each of the 168 reinforced concrete columns with respect to both ductile and fragile failure mechanisms, and thus estimating failure probability in a reliability-based format. Future developments will be oriented at understanding the role of uncertainties in the definition of the resistance model as investigated in Castaldo et al. (2020) on the final reliability results.

References

Baker, J.W., 2015. Efficient analytical fragility function fitting using dynamic structural analysis. Earthquake Spectra 31 (1), 579–599.

Castaldo, P., Gino, D., Bertagnoli, G., Mancini, G., 2020. Resistance model uncertainty in non-linear finite element analyses of cyclically loaded reinforced concrete systems. Engineering Structures, 211(2020), 110496,

Cornell, C., 1968. Engineering seismic risk analysis. Bulletin of the Seismological Society of America, 58 (5), 1583–1606.

DM 17/01/2018 (2018) Aggiornamento delle Norme Tecniche per le Costruzioni, Roma, Italy. (in Italian)

European Committee for Standardization, 2015. Eurocode 0: basis of structural design, Structural Eurocodes, 200290. Technical Committee CEN/TC 250.pectra 31 (1), 579–599.

INGV. Interactive Seismic Hazard Maps. Available at: http://esse1-gis.mi.ingv.it/s1_en.php (last access 04/03/2019).

Jalayer, F., Cornell, C.A., 2003. Direct probabilistic seismic analysis: implementing non-linear dynamic assessment. Stanford University.

Luzi, L., Puglia, R., Russo, E., 2016. ORFEUS, Engineering Strong Motion Database, Version 1.0. Istituto Nazionale di Geofisica e Vulcanologia, Observatories & Research Facilities for European Seismology, https://doi.org/10.13127/ESM. Available at: https://http://esm.mi.ingv.it/.

Mandre, J.B., Priestley, M.J.N., Park, R., 1988. Theoretical stress-strain model for confined concrete. Journal of Structural Engineering 114 (8), 1804–1826.

McGuire, R., 1995. Probabilistic seismic hazard analysis and design earthquakes: closing the loop. Bulletin of the Seismological Society of America, 85(5), 1275-1284.

Menegotto, M., Pinto, P.E., 1973. Method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending, Proceedings IABSE symposium of resistance and ultimate deformability of structures acted on by well-defined repeated loads, vol. 13. International Association of Bridge and Structural Engineering, Lisbon, Portugal, 15–22.

Ufficio del Genio Civile di Vicenza, 1948. La Ricostruzione del "Ponte della Vittoria" in Bassano del Grappa. Tipografia Minchio, Bassano del Grappa, pp. 12.

Vamvatsikos, D., Cornell, C.A., 2004. Applied incremental dynamic analysis. Earthquake Spectra, 20 (2), 523–553.

Zanini, M.A., Hofer, L., 2019. Center and Characteristic Seismic Reliability as new indexes for accounting uncertainties in seismic reliability analysis. Soil Dynamics and Earthquake Engineering, 123, 110-123.

Zanini, M.A., Hofer, L., Faleschini, F., Pellegrino, C., 2017. The influence of record selection in assessing uncertainty of failure rates. Ingegneria Sismica, 34 (4), 30-40.

Zanini, M.A., Hofer, L., Toska, K., 2019. A semi-analytical formulation for accounting uncertainties of hazard parameters in structural seismic reliability analysis. Engineering Structures, 192, 18-29.