Modeling of Masonry Bridges in Presence of Damage: The Case Study of San Marcello Pistoiese Bridge

Daniela Addessi[1], Domenico Liberatore[2], and Andrea Battisti[1](✉)

[1] Department of Structural and Geotechnical Engineering, Sapienza University of Rome, Via Eudossiana 18, 00184 Rome, RM, Italy
{daniela.addessi,an.battisti}@uniroma1.it

[2] Department of History, Representation and Restoration of Architecture, Sapienza University of Rome, Piazza Borghese 9, 00186 Rome, RM, Italy
domenico.liberatore@uniroma1.it

Abstract. Safeguard of the heritage of masonry infrastructural constructions is a relevant concern nowadays. The historical-monumental value of a large part of these artifacts, combined with their current functional reuse in the transport networks, make them interesting case studies. These structures are generally afflicted by structural weaknesses that make them vulnerable under dynamic actions, strongly present in the Italian peninsula, due to its widespread and high seismicity. In this framework, Finite Element analysis turns out to be a useful tool to better understand the structural behavior of masonry artifacts. In this paper, the seismic assessment of a masonry arch bridge, located in Italy, is proposed by means of a 3D FE modeling. The masonry material is described through the adoption of a constitutive law with damage, capable of capturing the degrading behavior of masonry under cyclic actions, characterized by a strain softening response. The phenomenological law is characterized by the presence of a scalar damage variable, describing the material degradation evolving during the analysis. After the investigation of modal shapes of the bridge, the horizontal capacity curve was estimated through a pushover analysis, and finally the structure was subjected to a set of natural accelerograms. The health state of the case study was consequently defined by means of damage indexes, and the most critical areas of the bridge were highlighted through the study of the damage patterns.

Keywords: Masonry Arch Bridge · Finite Element Modeling · Damage · Time History Analysis · Damage Index

1 Introduction

A growing interest from the scientific community is developing in the study of masonry bridges. This type of artifacts is indeed widely spread both in Italian and European countries, referring to multiple ages of construction. Nowadays, it is possible to find numerous examples of these structures, ranging from monumental walking bridges to

Y. Endo and T. Hanazato (Eds.): SAHC 2023, RILEM Bookseries 46, pp. 421–432, 2024.
https://doi.org/10.1007/978-3-031-39450-8_35

other few centuries old examples, which are currently re-used as part of road and rail transport networks. The interest in the study of these constructions, therefore, ranges from the field of architectural heritage conservation to infrastructural safety [1–3]. However, these are very vulnerable to seismic actions, due to the limited building knowledge of the past, and the numerous structural deficiencies. These latter are, in some respects, like those found in masonry buildings, such as the absence of transverse connections and the poor mechanical properties of the mortar adopted. In this framework, a survey made after 2016 Central Italy earthquake [4] focused the attention again on the issue of seismic vulnerability of Italian masonry bridges, showing the damage scenarios in several infrastructural constructions located in the epicenter area. Finite element (FE) analysis turns out to be a particularly useful tool to investigate masonry artifacts: it gives the possibility to carry out an extensive study that first allows to discover the structural deficiencies, thus performing a seismic vulnerability analysis, and then moves on to model possible reinforcement scenarios [5], obtaining a predictive evaluation of their effectiveness. Several approaches are now available to describe masonry response, such as micromechanical, macromechanical and multiscale models [6–8]. While modeling real-scale structures, a macromechanical modeling approach is employed. Accordingly, masonry heterogeneous material is treated as an equivalent homogeneous continuous medium, characterized by a phenomenological constitutive law whose parameters are derived by means of homogenization and identification techniques [3]. Classical constitutive laws are generally adopted to characterize materials, as commonly implemented in computational codes [9], but these are often not suitable to correctly reproduce the structural response of masonry in the nonlinear field. In the present work, the linear and nonlinear study of a masonry arch bridge is carried out, aiming at investigating its horizontal capacity and the structural damage activated under seismic actions. A constitutive law with damage is adopted, which models material degradation caused by tensile strain states, allowing global estimation of this by means of damage indexes and local-level assessment by means of damage patterns. The detailed formulation is contained in Addessi and Sacco 2016 [6]. The adopted model was originally formulated for micromechanical modeling of bricks, and later used for the macromechanical analysis of the “Ponte delle Torri” in Spoleto, showing that it is capable of reproducing, with good approximation, some of the damage scenarios currently present in the artifact [10, 11]. With the aim of estimating the structural response under seismic actions, a three-dimensional (3D) model of the bridge is developed, applying the three components of seismic actions and monitoring displacements and damage evolution history.

2 Description of the Bridge

The studied bridge, also known as the “Lizzanese” bridge, connects the towns of San Marcello Pistoiese and Lizzano, in Tuscany, crossing a natural riverbed. The artifact reasonably dates back to the 18th-19th centuries but underwent heavy reconstructions after World War II. The structure, regarding geometry and adopted materials, is strongly characteristic of many bridges present in central Italy, which makes its study even more significant for general concerns. In terms of geometry, the bridge has 3 arches, two symmetrical side arches of 8 m span and the central one of 21.5 m; the two central piers are founded in the riverbed, while the abutments follow the slope of the terrain. Overall, the structure has a length of 72.5 m, 5.8 m in cross-section, and an overall height from the base of the piers of 24.25 m including parapets. A large part of the construction is made of sandstone masonry, grouted with cement mortar in the external areas and lime mortar in the internal areas. The vaults are made of brick masonry and cement mortar, and the internal filling is in loose soil. Foundations are in reinforced concrete, over stratified rocks [12].

3 3D Modeling of the Structure

The bridge was modelled using the Finite Element Program FEAP [13], adopting 8-node 3D brick user-type elements, in which the constitutive law with damage described in [6] was implemented. 3 degrees of freedom are defined at each node, i.e., the three displacements u_k, v_k, w_k, and the displacement fields are interpolated by trilinear shape functions. The Gauss integration technique, with a 2x2x2 rule, is used to perform the required element computations. The bridge is fully restrained at the ground. Figure 1 shows the whole bridge and an exploded view separating the different materials, with reference to the discretization adopted for the analyses, consisting of 2724 FE. This mesh was selected to get a good match between accuracy of the results and computational burden. In Fig. 1 it is possible to notice the different materials employed and the way they are distributed in the structure: piers, spandrel walls, and abutments are made of sandstone masonry (red), the internal backfill is made of soil (yellow), and the vaults are made of brick masonry (blue).

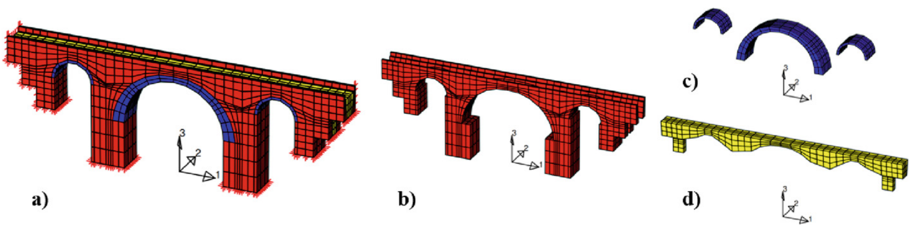

Fig. 1. 3D model of the bridge (a); exploded view of the adopted materials: sandstone masonry (b), brick masonry (c), backfill (d).

3.1 Damage Law Formulation

The adopted constitutive law was presented in Addessi and Sacco 2016 [6], and already applied in the field of macromechanical modeling of masonry bridges for the study of "Ponte delle Torri", in Spoleto. This latter was first subjected to a seismic assessment [10] and then, underwent a simulation of a seismic sequence adherent to the main events that involved the structure during its life [11], obtaining good agreement with the current damage state. The constitutive model is meant to describe damage related to microfractures process for prevailing tensile strain states. The formulation involves the use of a single scalar damage variable D, acting equally on each term of the elastic matrix and modeling isotropic damage. The relationship is as follows:

$$\boldsymbol{\sigma} = (1 - D)\mathbf{E}\,\boldsymbol{\varepsilon} \tag{1}$$

with $\boldsymbol{\sigma}$ denoting the 6-component stress vector, $\mathbf{E}$ the 6x6 elastic constitutive matrix, and $\boldsymbol{\varepsilon}$ the 6-component strain vector. Damage onset and evolution is driven by an associated variable, which measures an equivalent strain state. This is defined on the basis of the positive principal strains $\varepsilon_{i/j,}$ also accounting for the effects of negative principal strains. The parameter ε_0 is a regularization factor that ensures the convexity of the limit domain and k regulates the effect of the negative strains. Therefore, the definition of the equivalent strain measure is given by:

$$\varepsilon_{eq} = \sqrt{\langle\sum_{i=1}^{3}\langle\varepsilon_i + \varepsilon_0\rangle_+^2 - k\sum_{i=1}^{3}\sum_{j=1}^{3}\frac{(1-\delta_{ij})}{2}\langle\varepsilon_i\rangle_-\langle\varepsilon_j\rangle_-\rangle_+} - \varepsilon_0 \tag{2}$$

To overcome the mesh dependency problems arising in presence of strain-softening materials, such as masonry, the nonlocal integral regularization technique is here adopted. This involves the use of a weighting function ψ with Gaussian shape, which permits to account for the influence, on each Gauss point, of the average strain state occurring at points lying in its neighborhood. The size of this latter is regulated by the nonlocal radius, defined based on the masonry size. Then, the equivalent strain is defined as the following nonlocal measure:

$$\overline{\varepsilon}_{eq}(\mathbf{x}) = \frac{1}{\int\psi(\mathbf{y})dV}\int\psi(\mathbf{x}-\mathbf{y})\varepsilon_{eq}(\mathbf{y})dV \tag{3}$$

The adoption of the nonlocal regularization technique allows to overcome the mesh-dependency of the FE solutions leading to objective numerical results.

The damage evolution law uses the nonlocal strain measure and depends on the following mechanical parameters: β governs the shape of the softening branch, ε_t the tensile strain threshold, and ε_u the value of the equivalent strain corresponding to the completely damaged state. For a detailed discussion of the parameters, refer to [6, 10, 14].

$$\tilde{D} = 1 + \frac{1}{\overline{\varepsilon}_{eq}(\varepsilon_t - \varepsilon_u)^3}e^{-\beta(\overline{\varepsilon}_{eq}-\varepsilon_t)}\left(\overline{\varepsilon}_{eq} - \varepsilon_u\right)^2\left(\overline{\varepsilon}_{eq}\varepsilon_u + \overline{\varepsilon}_{eq}\varepsilon_t - 2\varepsilon_t^2\right) \tag{4}$$

Damage estimation during the analyses, at each time step, is performed according to the following relation:

$$D = \max_{history}\left\{0, min\left\{\tilde{D}, 1\right\}\right\} \tag{5}$$

also considering the assumption of thermodynamics irreversibility of damage, for which $\dot{D} \geq 0$. The physical variation range of the damage scalar variable D is between 0 and 1, corresponding to the undamaged and fully damaged condition.

3.2 Modal Analysis

The bridge model was first validated in the linear elastic field to test the efficiency of the geometric modeling and restraint scheme. The modal analysis was performed. Mode 1 and mode 5, representing out-of-plane and vertical mode, respectively, obtained with the FE model were compared to the data shown in literature [9, 12, 15]. The bridge, already studied in literature, was modeled by means of other computational software and an in situ dynamic investigation, with accelerometers, was also performed. Sandstone masonry was characterized by a Young's modulus E of $10 \mathrm{x} 10^9$ N/m^2 and a material density γ of 2200 kg/m^3; the brick masonry was characterized by a Young's modulus of $12 \mathrm{x} 10^9$ N/m^2 and a material density γ of 1800 kg/m^3; for the backfill a Young's modulus of $5 \mathrm{x} 10^9$ N/m^2 and a γ of 1800 kg/m^3 were assumed. For all the materials, a value of Poisson ratio ν equal to 0.2 was adopted. The chosen FE discretization, made of 2724 FEs was defined after an updating process within modal analysis, reaching a good match with literature benchmarks. The same mesh was kept in the following nonlinear analyses, where it showed good accuracy and efficiency properties, despite the heavy computational burdens. Periods and frequencies are given in Table 1, and Fig. 2 shows the associated modal shapes.

Table 1. Periods and frequencies for mode 1 and mode 5: FEAP model vs Experimental.

Modal analysis	FEAP model T [s]	FEAP model f [Hz]	Experimental f [Hz]	Variation of f %
Mode 1 (out-of-plane)	0.23	4.21	3.998	5.3
Mode 5 (vertical)	0.07	14.50	13.911	4.2

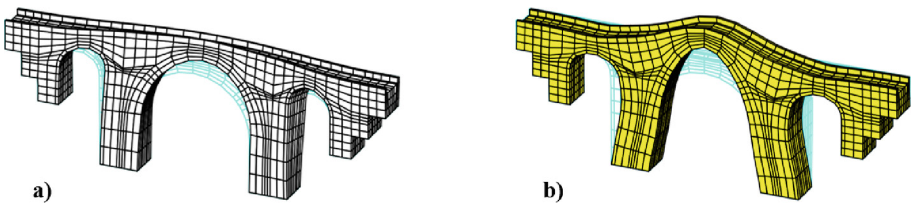

Fig. 2. Modal shapes: Out-of-plane mode 1 (a), vertical mode 5 (b).

4 Nonlinear Analyses

After the modal validation, nonlinear analyses were performed, starting with a pushover followed by time history analyses. In all these analyses, the backfill was considered as a non-structural material, assigning to it a linear elastic constitutive law so that it does not influence the damage scenario of the structure during the analyses. Performing analyses in the nonlinear field, and considering the uncertainties in the moduli of materials, as well as possible ageing phenomena, it was considered appropriate to reduce Young's moduli. Similar considerations are given in [12, 15]. Backfill therefore results in the same values of ν and γ as mentioned before, and the Young's modulus E equal to $5\text{x}10^8$ N/m^2. Regarding the structural materials, sandstone and brick masonry, the same Poisson ratio ν and γ mentioned above were adopted, while the tensile and compressive thresholds were deduced from the Italian Guidelines NTC [16]. Sandstone masonry was characterized by a compressive strength f_c equal to 8.5 MPa and tensile strength f_t equal to 0.55 MPa, brick masonry by f_c equal to 4.2 MPa and f_t equal to 0.3 MPa. Figure 3 shows the damage laws adopted for the structural materials, and Table 2 shows the related set of parameters.

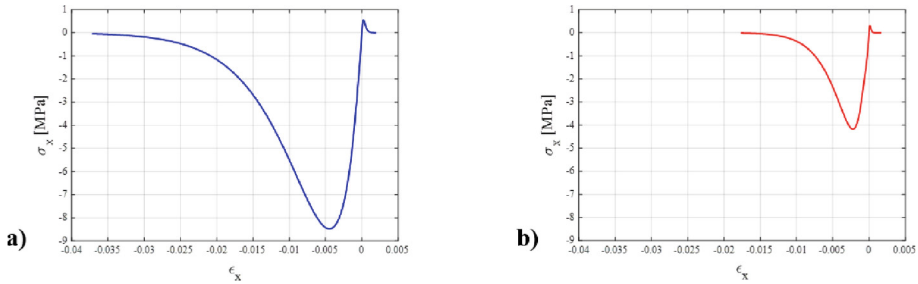

Fig. 3. Constitutive laws with damage: sandstone masonry (a), brick masonry (b).

Table 2. Parameters adopted for the damage law of sandstone masonry and brick masonry.

Material	E [N/m^2]	ν	ε_t	k	ε_u	ε_0	β_t	β_c	α
Sandstone masonry	$5\text{x}10^9$	0.2	$0.4\text{x}10^{-4}$	0.03	$5\text{x}10^{-2}$	10^{-5}	5000	750	6000
Brick masonry	$6\text{x}10^9$	0.2	$0.05\text{x}10^{-4}$	0.03	$500\text{x}10^{-2}$	10^{-5}	12000	1800	4000

4.1 Pushover Analysis

Considering the structural symmetry of the bridge, restraints and loading conditions, the pushover analysis was conducted on a half-bridge model (1362 FE), reducing the computational burden. The analysis was performed by first applying the self-weight and then, a distribution of horizontal mass proportional forces. The structural response was monitored through two control nodes: a point on the road surface at the center of the central arch, at centerline, called TOP, and a point at the height of the center of mass called CM, at an elevation of 14 m from the base of central pier, at centerline. The arc-length solution algorithm was adopted, as it allows to follow the post-peak softening branches. This behavior is due to the strain softening of the constitutive laws employed, which well-represent the real behavior of masonry. The curve presents a first elastic branch, then a peak zone corresponding to the maximum base shear and, finally, a softening branch with loss of strength and increase of displacement. Given the adopted laws, as expected, the shape of the curve and the maximum force and displacement values differ from those evaluated in [12], where the authors adopted elastic-plastic laws without damage. As obvious, the reading at the TOP control node provides a higher displacement compared to the CM, as also highlighted in [9], where the issue of the choice of control points for bridges is discussed. The displacement monitoring of point CM thus results in a more conservative capacity curve, with the same maximum base shear but lower subtended area. Figure 4 shows the pushover curves and damage patterns detected at the end of the analysis: structural damage is highlighted in the crowns of the arches, at the base of the pier, and mainly in the abutment zone of the pushing side, where tensile states are more concentrated.

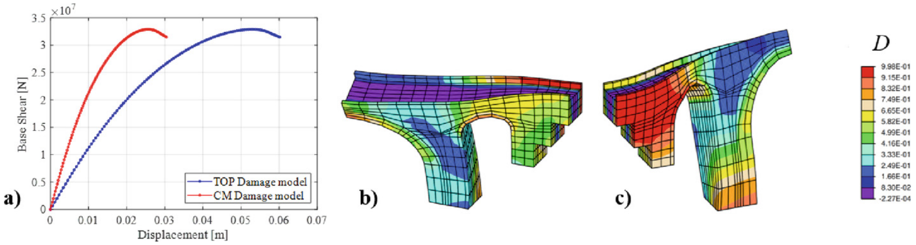

Fig. 4. Pushover curve for TOP and CM control nodes (a); Damage patterns at the final step of the analysis: South side view (b), North side view (c).

4.2 Time History Analysis

A time history analysis campaign was performed relying on the study of the seismic history of the site where the bridge is situated. The investigations were articulated starting from the Italian Macroseismic Database DBMI [17], where the major events in the municipality of San Marcello Pistoiese were found and completed with data for the adjacent town of Pistoia. By means of the Italian Accelerometric Archive ITACA [18], the main fault mechanisms found in recent seismic events of the area were identified. A subsequent phase followed, searching for natural records that had characteristics related to the seismic history of the site and using the Engineering Strong Motion Database [19]. A set of 7 events was created, and the spectral compatibility procedure was performed in the period range between 0.1 and 1 s, taking care to keep the scaling factors as small as possible, not to excessively alter the signal. The set has events with a M_w value between 5.6 and 6.5, epicentral distance less than 30 km, and focal mechanisms of normal or thrust type. To perform analyses well matching the real conditions, the three components of earthquake N-S, E-W, Z were considered simultaneously. In addition, 3 return periods were considered: 201, 475, 975 years for the purpose of investigating increasing damage scenarios. In this process, each component was scaled regarding its relative response spectrum from NTC Code [16], for each return period. N-S and E-W components were scaled with respect to the horizontal response spectrum, Z component with respect to the vertical response spectrum of the site. Figure 5 shows the spectral compatibility procedure for the N-S direction, assigned along the out-of-plane direction of the bridge, for the 3 return periods. E-W component was assigned along the longitudinal direction of the structure, Z component was assigned along the vertical direction. A Rayleigh damping method was adopted, considering the first two angular frequencies of the structure and a 5% damping ratio. A Newmark implicit integration with β and γ equal to 1/4 and 1/2, respectively, is used to solve the dynamic problem, combined with a Newton-Raphson algorithm for the solution of the non-linear behavior.

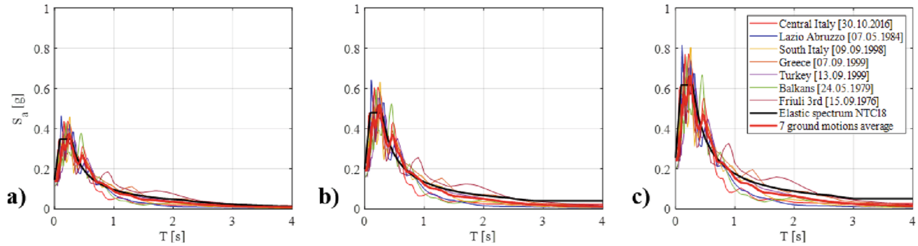

Fig. 5. Elastic spectra of the 7 ground motions selected, NTC18 elastic spectrum for San Marcello Pistoiese site (black), average spectrum of the 7 ground motions set (red): 201 years return period (a), 475 years return period (b), 975 years return period (c).

The same group of analysis was carried out for both the damage and elastic model of the entire bridge, in order to compare the responses. Analyzing the maximum displacements for the out-of-plane direction, read at the TOP point, where the most significant displacements are displayed, and comparing the three return periods and the two models (Fig. 6), some interesting observations can be made. As for the 201 years return period, four out of seven events with maximum displacement reached by the elastic model are observed. This scenario varies considerably for the 975 years return period graph, where five out of seven events present a maximum response shown by the damage model. Examples of this are the Greece 1999 and South Italy 1998 events, where it is possible to notice this amplification in the damage model response for 975 years return period. The bridge experiencing damage, thus varying its modal periods, will experience new resonance conditions towards the seismic action.

As an example, for the Greece 1999 event with 975 years return period, all the results of the analysis are shown in Fig. 7. The greatest displacements, as expected, are reached along the out-of-plane direction, where an amplification phenomenon is observed in the response evaluated by the damage model. The responses along the E-W and Z directions are considerably less significant. It is interesting to note, along the Z direction, how the oscillations start from an initial value of attestation, then stabilize on a new value corresponding to a residual displacement at the end of the analysis. The transition between the initial and residual displacement has a trend closely related to the growth of damage in the structure. This latter is highlighted, for the damage model, through damage indexes, evaluated at each time step of the analyses. The Global Damage Index (*GDI*) [10] and its modified version based on fractile at 95%, ($D_{0.95}$) [11], were implemented by the authors. Regarding the damage patterns, it is possible to track the

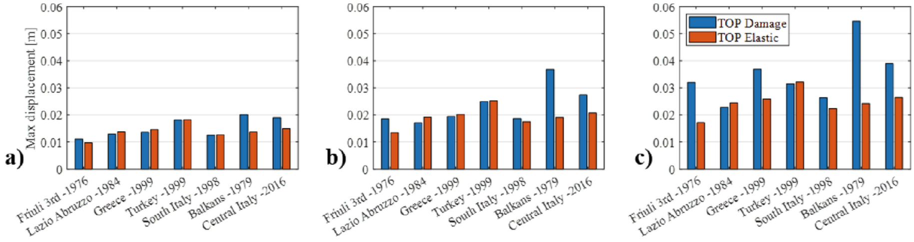

Fig. 6. Maximum displacements of the TOP point along the out-of-plane direction (N-S) for damage and elastic models: 201 years return period (a), 475 years return period (b), 975 years return period (c).

damage evolution. Referring again to the Greece 1999 event in case of 975 years return period, at the end of the analysis, a damage pattern with an approximately symmetrical damage distribution is obtained. Symmetry is broken only locally near the spandrel walls above the west arch, where a peak of the damage can be detected. Maximum damage is localized around the crowns of the three arches. In other events, such as the Turkey 1999, for the same return period, non-symmetrical damage pattern could be observed, indicating that the E-W components has a non-negligible effect in the directionality of damage propagation. Moreover, the bridge experiences an excitation of the higher modes.

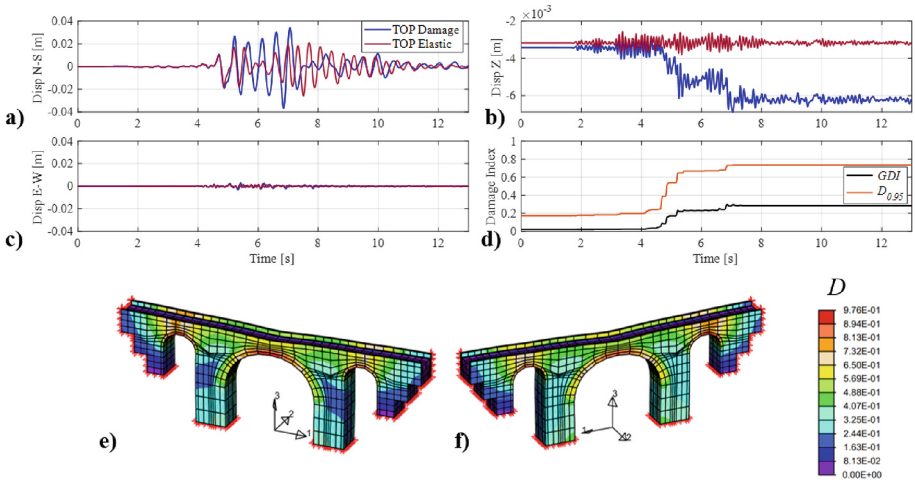

Fig. 7. Response of the bridge under Greece 1999 record with 975 years return period: N-S displacement (a), Z displacement (b), E-W displacement (c), damage indexes (d), damage pattern south side view (e), damage pattern north side view (f).

More investigations regarding dynamic amplification were conducted by means of the Fast Fourier Transform (FFT) of the displacement response (Fig. 8). By evaluating the damaged period through frequency peaks and highlighting this value into the scaled N-S displacement spectrum of the earthquake, good agreement was found with the response reported before. The damage model shows an increased period compared to that of the elastic model. However, the displacement spectrum refers to one direction only, that out-of-plane for the bridge, and therefore to only one predominant modal form, the first one. However, the structure is subjected to accelerations along the three directions experiencing multiple modes excitation, resulting in a simplified procedure for the study of the phenomenon. This explains the variation between the maximum displacement shown in Fig. 7 and what is evaluated using the spectrum in Fig. 8.

For all the analyses performed adopting the damage model, the evolution of damage in the bridge is also followed by means of the damage indexes mentioned above. A summary graph of the $D_{0.95}$ index for the analyses with 975 years return period is shown in Fig. 9. This index, by evaluating the average damage identified in the structural volume, turns out to be a synthetic measure of the structural health, showing during the time of the analysis the progression of degradation. It is worth noting the different gradient through which the various events reach the threshold of maximum damage; more impulsive earthquakes explicate in few instants the total damage, as in the case of Southern Italy 1998 or Balkans 1979 events. Once the maximum peaks in earthquake acceleration are attained, given the condition of thermodynamic irreversibility, damage reaches the maximum value and remains constant until the end of analysis. Regarding the set employed, the most severe earthquakes, which reach the most severe degradation, turned out to be Balkans 1979 and Central Italy 2016, with an index value higher than 0.8.

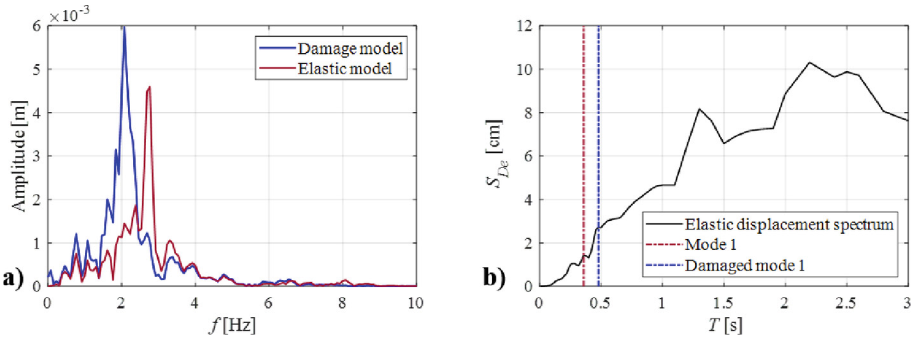

Fig. 8. Fast Fourier Transform (FFT) of the out-of-plane displacement for the damage and elastic models for Greece 1999 record with 975 years return period (a); elastic and damaged mode 1 period compared to the elastic displacement spectrum (b).

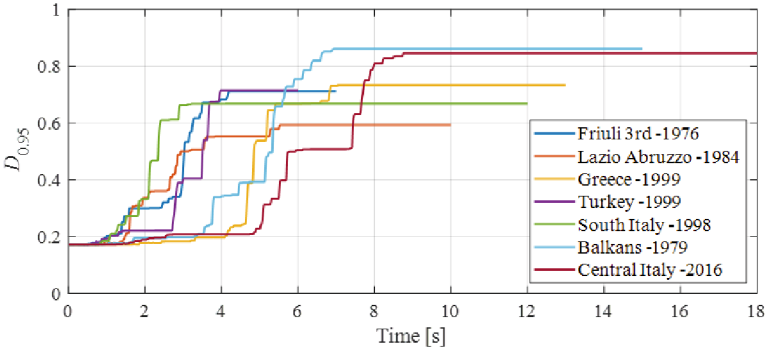

Fig. 9. Damage index $D_{0.95}$ for the 7 records with 975 years return period.

5 Conclusions

The present study adopted a macromechanical constitutive law with damage to study the static and dynamic nonlinear response of a masonry arch bridge. The isotropic phenomenological law, characterized by the introduction of one scalar damage variable, is quite reliable in reproducing the behavior of masonry under seismic loading conditions. This formulation, after initial applications in the micromechanical framework, and subsequent extensions to the macromechanical framework, sees in this work a further field of testing. The study of a three-arch masonry road bridge, with a recurring construction scheme for small-sized historical bridges in Italy, is definitely useful for both in-depth investigation of the structural response and comparative analyses with other modeling choices for similar case studies.

Adopting a 3D FE model and accounting for seismic action in the three directions, an intent was made to reproduce realistic conditions as accurately as possible to obtain a more reliable response. Therefore, after modal validation of the model and estimation of the horizontal capacity with pushover curve, an investigation at different damage levels, considering three return periods, employing a set with heterogeneous characteristics, stands as a fundamental tool to survey the structural weaknesses. Among the potentialities of the FEAP user finite element, where the formulation was implemented, is the ability to show damage maps, relevant to identify areas where damage is localized during the earthquake, and thus being able to hypothesize incipient collapse mechanisms.

Moreover, being able to follow the progression of damage by means of damage indexes, allows a view of the overall damage on the structure, and thus shows how severe an earthquake is, with respect to the bridge in terms of the velocity of damage evolution and maximum damage reached. Performing a comparison with an elastic model of the bridge for the time history analyses, offered the opportunity for evaluations of the dynamic amplification of the response, which in most cases occurred for the damage model.

Future studies could see interest in modeling structural reinforcement techniques designed to repair the most vulnerable areas of the structure, as well as more in-depth investigations into the incidence of the Z component for earthquakes in near-fault events.

References

1. Gönen, S., Soyöz, S.: Seismic analysis of a masonry arch bridge using multiple methodologies. Eng. Struct. **226**, 111354 (2021). ISSN 0141-0296
2. Roselli, I., et al.: Health assessment and ambient vibration testing of the "Ponte delle Torri" of Spoleto during the 2016–2017 Central Italy seismic sequence. J. Civ. Struct. Heal. Monit. **8**(2), 199–216 (2018). https://doi.org/10.1007/s13349-018-0268-5
3. Karaton, M., Aksoy, H. S., Sayın, E., Calayır, Y.: Nonlinear seismic performance of a 12th century historical masonry bridge under different earthquake levels. Eng. Failure Anal. **79**, 408–421 (2017). ISSN 1350-6307
4. Di Sarno, L., da Porto, F., Guerrini, G., Calvi, P.M., Camata, G., Prota, A.: Seismic performance of bridges during the 2016 Central Italy earthquakes. Bull. Earthq. Eng. **17**(10), 5729–5761 (2018). https://doi.org/10.1007/s10518-018-0419-4
5. Gattulli, V., Lofrano E., Paolone A., Pirolli G.: Performances of FRP reinforcements on masonry buildings evaluated by fragility curves. Comput. Struct. **190**, 150–161 (2017). ISSN 0045-7949
6. Addessi, D., Sacco, E.: Nonlinear analysis of masonry panels using a kinematic enriched plane state formulation. Int. J. Solids Struct. **90**, 194–214 (2016)
7. Di Re, P., Addessi, D., Sacco, E.: A multiscale force-based curved beam element for masonry arches. Comput. Struct. **208**, 17–31 (2018)
8. Addessi, D., Di Re, P., Gatta, C., Sacco, E.: Multiscale analysis of out-of-plane masonry elements using different structural models at macro and microscale. Comput. Struct. **247**, 106477 (2021)
9. Pelà, L., Aprile, A., Benedetti, A.: Comparison of seismic assessment procedures for masonry arch bridges. Construct. Build. Mater. **38**, 381–394 (2013). ISSN 0950-0618
10. Addessi, D., Liberatore, D., Nocera, M.: Damaging Behavior of Masonry Arch Bridges: Analysis of 'Ponte delle Torri' in Spoleto, Italy. Journal of Earthquake Engineering (2020)
11. Addessi, D., Gatta, C., Nocera, M., Liberatore, D.: Nonlinear dynamic analysis of a masonry arch bridge accounting for damage evolution. Geosciences **11**, 343 (2021)
12. Pelà, L., Aprile, A., Benedetti, A.: Seismic assessment of masonry arch bridges. Eng. Struct. **31**(8), 1777–1788 (2009)
13. Taylor, R.L.: FEAP—A Finite Element Analysis Program, Version 8.5; Department of Civil and Environmental Engineering, University of California at Berkeley, Berkeley, CA, USA (2017)
14. Addessi, D., Sacco, E.: Enriched plane state formulation for nonlinear homogenization of in-plane masonry wall. Meccanica **51**(11), 2891–2907 (2016). https://doi.org/10.1007/s11012-016-0484-1
15. Aprile, A., Pelà, L., Benedetti, A.: Analisi pushover di ponti in muratura. PONTI E VIADOTTI: ASPETTI PROGETTUALI, STRUTTURALI E DI MANUTENZIONE (2006). [in Italian]
16. NTC 2018: Italian Building Code for Constructions (Norme Tecniche per le Costruzioni). D.M. 17th January 2018 – S.O. n. 8, G.U. n. 42; 20th February 2018. [in Italian]
17. Locati, M., Camassi, R., Stucchi, M.: DBMI11, la versione 2011 del Database Macrosismico Italiano. Milano, Bologna. Accessed September 12 (2018). http://emidius.mi.ingv.it/DBMI11
18. Russo, E., et al.: Italian Accelerometric Archive v3.2 - Istituto Nazionale di Geofisica e Vulcanologia, Dipartimento della Protezione Civile Nazionale (2022)
19. Luzi, L., Puglia, R., Russo, E., Orfeus, W.: Engineering Strong Motion Database; Version 1.0; Istituto Nazionale di Geofisica e Vulcanologia, Observatories & Research Facilities for European Seismology: Roma, Italy, vol. 10 (2016)