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FINITE ELEMENT MODELING OF AN ITALIAN MASONRY ARCH BRIDGE DETECTING DAMAGE UNDER SEISMIC EXCITATIONS

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Abstract

The architectural heritage of masonry constructions, widely spread in the Italian country, is extremely vulnerable when compared to the high level of seismicity of the peninsula. Most of this heritage is nowadays still characterized by unreinforced historical masonry which due to deterioration phenomena caused by ageing, the changing of use conditions, the dated construction and design criteria, is already showing active damage processes. These conditions of decay and vulnerability, under earthquake can lead to major collapse scenarios. A reliable modeling of masonry material response becomes essential to understand and prevent such damage phenomena in the structures. To achieve this purpose there is a need to adopt constitutive laws accounting for damage, plasticity and other main phenomena, thus to be able to capture the strain softening behavior of masonry material in seismic conditions. Relying on these grounds, the following paper presents the study conducted on an arch bridge, located in Italy, built with a traditional stone masonry. Through modal analysis, pushover and time history analysis campaign, the structural response of the structure is investigated. The 3D modeling of the structure is carried out by means of the finite element software FEAP, characterizing masonry through the constitutive damage law Addessi and Sacco 2016, within a macromechanical modeling. In addition to monitoring the most usual engineering demand parameters (EDPs), the implemented procedure allows to follow the evolution of decay by means of damage maps and damage indexes, useful tools for an accurate insight into the vulnerabilities of the bridge when subjected to horizontal actions.

Keywords: Damage, Finite Element Modeling, Masonry Arch Bridge, Seismic Assessment

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1 INTRODUCTION

The study of civil masonry constructions is still one of the main open challenges in the world of academic research in the engineering field. Given the complex heterogeneity of the composite material, made by the combination of unit and mortar, and given the widespread variability of the mechanical properties of these components, it is not easy to univocally characterize masonry behavior in numerical modeling. Finite Element analysis is one of the most widely adopted methods to study masonry constructions, and depending on the scale of investigation, micromechanical modeling is generally performed on small portions of masonry whereas macromechanical modeling is preferred for entire artifacts. Multiscale approaches give a good compromise between accuracy and computational efficiency [1]. By varying the scale of investigation, it is possible to detect a local and detailed structural response in the former case, and then move on to assess a global behavior in the latter. In the framework of macromechanical modeling, masonry is reviewed as an equivalent homogeneous continuous material, characterized by a phenomenological constitutive law [2]. By means of proper homogenization and identification techniques it is possible to determine the main mechanical parameters of the homogenized material; however, of fundamental importance remains the choice of the constitutive law to be employed. Commercial codes generally offer the option of adopting classical constitutive laws such as elastic and elastic-plastic [3] which, however, do not reliably reproduce the response of masonry when subjected to cyclic actions. Constitutive laws with damage, damage and plasticity, no tension are instead capable of adequately capturing the strain softening of the material in the nonlinear field going to contemplate, in relation to the complexity of the formulation and the number of variables introduced, different phenomena of interaction and degradation occurring at the joints or in the individual constituents [4, 5]. A reliable modeling of masonry material makes it possible to perform entire building studies that are as close as possible to accurately reproduce the real behavior, especially in the seismic field. In recent years there has been increasing attention to the study of historical masonry bridges, which turn out to be complex structures in geometry and shape, but characterized by constructive approaches and structural vulnerabilities quite similar to those of traditional ancient masonry buildings. The study of bridges is definitely important from the perspective of their preservation over time; numerous examples are found in the Italian territory, either as architectural monuments or re-integrated constructions into modern transportation system and therefore subject to new loading conditions [6, 7]. The frequent seismic events in Italy, however, call for the need of a reduction of structural weaknesses, which are still a problem for these structures that under high horizontal loads show major damage scenarios [8]. The present work shows the study of vulnerability for a masonry arch road bridge by means of three-dimensional (3D) Finite Element modeling performed in FEAP software [9], using a macromechanical approach, and adopting the damage law for masonry proposed by Addessi and Sacco 2016 [4]. After the model validation in the elastic field, the bridge was investigated in its damage scenarios by nonlinear analyses, highlighting its most critical regions. Damage was assessed in global terms by means of damage indexes and damage maps. Performing a suitable seismic assessment is essential to highlight major structural deficiencies, which will be restored in the following retrofit phase.

2 STRUCTURAL DESCRIPTION AND FE MODELING OF THE BRIDGE

The three-arch masonry bridge presented in this paper is located in the municipality of San Marcello Piteglio, in Tuscany, Italy, and connects two towns in the Apennine area, in the province of Pistoia. The construction has two minor lateral arches with a span of 8 m and a main central one of 21.5 m. The overall length of the structure is 72.5 m, the overall height is 24.25 m including the parapet, and there is a constant cross width of 5.8 m. The bridge has highly recurring constructive features and a structural scheme for Italian bridges dating around the 19th century, but over time the structure has undergone reconstruction works and is currently re-used in the road network. The resisting structure is realized in sandstone masonry used in piers, spandrel walls, abutments, while the vaults are made of brick masonry. The inside of the structural envelope features a backfill made of soil extending from street to vaults level. Different bonding mortars can be found in the bridge: the exterior facades and vaults present cement mortar, the inner zones of the sandstone masonry are bonded with lime mortar; the bridge is founded on stratified rocks [10].

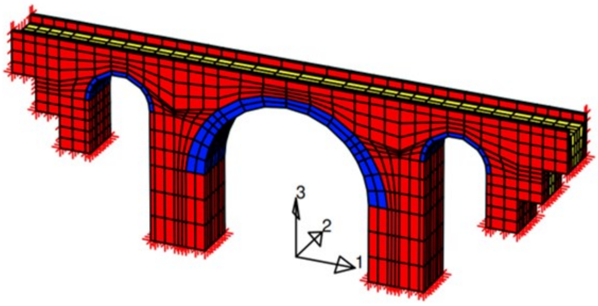

Figure 1: 3D FE FEAP model of the bridge.

Figure 1 shows the three-dimensional (3D) model realized in FEAP software [9], where it is possible to distinguish by different colors the various materials employed. In red the sandstone masonry, in blue the brick vaults and in yellow the backfill. The discretization adopted in all the conducted analyses is 2724 FE, where this value has been defined to give a good compromise between model accuracy and computational times. The model was developed employing 8-node user brick elements, where the authors implemented the constitutive law with damage presented in Addessi and Sacco 2016 [4]. Each node is equipped with three degrees of freedom, the three nodal translations u_k, v_k, w_k, and the displacement fields are interpolated by trilinear shape functions in the element domain. The Gauss integration technique, with a 2x2x2 rule, has been adopted in the FE procedure. Finally, a full restraint scheme has been used for the whole bridge model, at the base of the piers and abutments, and along the external vertical sides of the abutments.

2.1 Damage law for masonry

Masonry material response is described by the constitutive law with damage proposed in Addessi and Sacco 2016 [4]. This phenomenological law has already been applied in FE analyses of masonry bridges, within a macromechanical approach, as reported in [6, 11] where the response of the monumental bridge of Spoleto, in Italy, has been extensively investigated under seismic actions focusing on its damage scenarios. The adopted isotropic law models the damage evolution due to microfracture process related to prevalent tensile states in masonry. A scalar damage variable D is introduced in the formulation which equally influences all the terms of the elastic constitutive matrix $\mathbf{E}$. The following relation is adopted:

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

with **σ** and **ε** the 6-component stress and strain vectors, and **E** the 6x6 elastic constitutive matrix.

An associated variable governs the damage evolution; this is an equivalent strain measure whose formulation is based on principal tensile strains, also taking into account recovery for compressive states via the parameter k. The convexity of the limit domain is ensured by the regularization factor ε_0. This is defined as:

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

The nonlocal integral regularization technique has been adopted by the authors to overcome mesh dependency problems experienced in the solution of finite element problems when materials with strain softening behavior, such as masonry, are employed.

The nonlocal formulation of the equivalent strain measure results as follows:

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

where the presence of the weighting function ψ takes into account the influence of the surrounding Gauss points on the analyzed point through a Gaussian shape function. Damage evaluation at each step of the analysis is carried out as expressed in the following relation:

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

(4)where $\tilde{D}$ is the current damage value evaluated in the basis of the nonlocal equivalent strain measure and several mechanical parameters, for which a more detailed description is reported in [4, 11, 12]:

$$\tilde{D} = 1 + \frac{1}{\bar{\varepsilon}_{eq}(\varepsilon_t-\varepsilon_u)^3}e^{-\beta(\bar{\varepsilon}_{eq}-\varepsilon_t)}\left(\bar{\varepsilon}_{eq}-\varepsilon_u\right)^2\left(\bar{\varepsilon}_{eq}\varepsilon_u + \bar{\varepsilon}_{eq}\varepsilon_t - 2\varepsilon_t^2\right) \tag{5}$$

Damage formulation is also based on the assumption of thermodynamic irreversibility, for which $\dot{D} \geq 0$, and the maximum damage state is reached for a value of the scalar variable D equal to 1.

3 FE ANALYSES OF THE BRIDGE

Considering the described bridge FE model, linear and nonlinear analyses were conducted. First a modal analysis was performed to validate the geometry and restraint scheme with respect to the benchmarks available in literature [10, 13]. After that, the model was subjected to nonlinear analyses, first determining the horizontal capacity curve through pushover analysis and then performing a seismic assessment by using natural accelerograms derived from a research on the seismic history of the site.

3.1 Modal analysis

As for the sandstone masonry the Young's modulus E equal to $10\text{x}10^9$ N/m^2 and the material density γ equal to 2200 kg/m^3 were adopted, while as for the brick masonry the Young's modulus equal to $12\text{x}10^9$ N/m^2 and the material density γ equal to 1800 kg/m^3 were set. As for the backfill the Young's modulus is set equal to $5\text{x}10^9$ N/m^2, γ = 1800 kg/m^3 and the Poisson ratio ν = 0.2 was assigned to all the materials. When considering the model made of 2724 FE, a slight variation in terms of frequencies from experimental values was recorded, a result that is improved by increasing discretization level. On the overall, modal analysis showed a good agreement with respect to experimental data present in literature [10, 13] where the frequencies of the out-of-plane mode (mode 1) of the bridge and the vertical mode (mode 5) are reported, detected by means of an in situ survey. Table 1 shows the results of the modal analysis.

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

Table 1: Comparison of modal analysis results for FE model vs Experimental outcomes.

3.2 Pushover analysis

For all the nonlinear analyses performed, sandstone masonry and brick masonry are assumed as structural materials while the backfill as non-structural material. Therefore, the first two materials were characterized by the mentioned constitutive damage law, while a linear elastic law was adopted for the backfill in order not to affect the damage scenarios of the bridge. For the following nonlinear analyses, it was considered appropriate to perform a conservative reduction of the elastic moduli, as also reported in [10, 13]. Therefore, the Young's modulus equal to $5\text{x}10^8$ N/m^2 was assigned to the backfill.

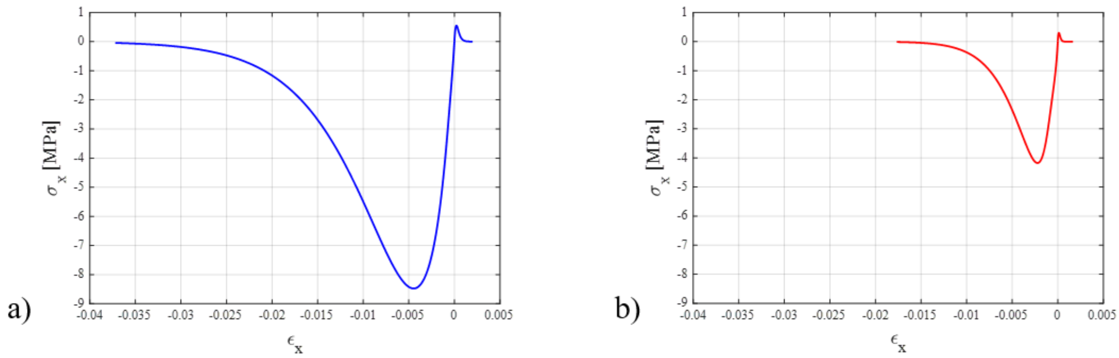

Figure 2: Damage constitutive laws for sandstone (a) and brick (b) masonry.

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

Table 2: Sandstone and brick masonry parameters for the constitutive damage laws.

Structural materials were characterized with the constitutive laws shown in Figure 2, whose parameters are reported in Table 2. Tensile and compressive strength thresholds were determined from the Italian Guidelines NTC [14]. It is assumed f_t equal to 0.55 MPa and f_c

equal to 8.5 MPa for sandstone masonry , f_t equal to 0.3 MPa and f_c equal to 4.2 MPa for brick masonry, in agreement with classical f_c/f_t ratios adopted for masonry.

The pushover analysis was conducted in two phases, first applying the self-weight of the structure and then performing an incremental analysis with out-of-plane forces having a mass proportional distribution. Taking advantage of the structural symmetry, the analysis was conducted on a half-bridge model made of 1362 FE. The structural response was monitored through two control nodes: the TOP and CM, both positioned in the centerline of the bridge. The TOP point is at the level of the road surface above the crown of the central main arch, and the CM point is at the level of the center of mass, at a height of 14 m from the base of the structure, positioned between the main and the lateral arches. The obtained horizontal capacity curve is shown in Figure 3. It is possible to observe an initial linear elastic section, a peak zone and a softening branch with loss of strength and increase of displacement, a response closely related to the strain softening behavior of the masonry material. It was possible to capture this behavior through the adoption of the aforementioned damage law, and through the use of the arc-length solution algorithm in the procedure. The capacity curve evaluation obtained varies in relation to the control node adopted: at the same value of maximum base shear the response read at the CM offers a conservative solution in terms of displacement. The obtained results, given the different laws adopted, show differences with those reported in [10], where the authors adopted elastic-plastic laws for the materials.

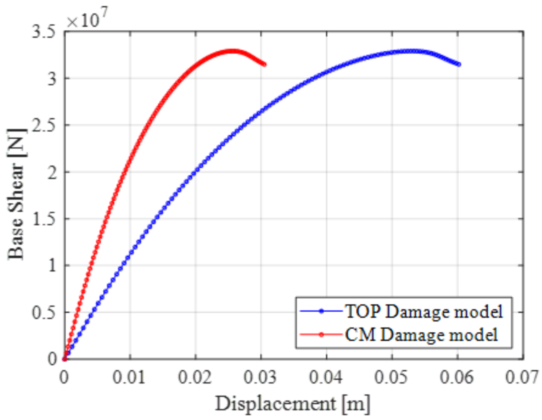

Figure 3: Pushover curve for TOP and CM control nodes.

3.3 Time history analysis

A time history analysis campaign was performed for the entire bridge model. Specifically, a set of 7 natural records was created, considering 3 return periods, 201, 475 and 975 years, with the aim of evaluating increasing levels of damage on the structure. The three components of earthquake acceleration were scaled in a range between 0.1 and 1 seconds, to have a good fit between the average spectrum of the set and the elastic spectra from NTC Code [14] along the three respective directions, considering the relative spectrum for each direction and aiming to have scaling factors as contained as possible. The choice of performing analyses with the three earthquake components was made with the purpose of trying to reproduce as realistically as possible the condition of the structure under earthquake in real events. Specifically, the N-S component was applied in the out-of-plane direction of the bridge, the E-W along the longitudinal and the Z along the vertical direction. The set presents events characterized by focal mechanisms of normal or thrust type, with M_w values between 5.6 and 6.5, and with an epicentral distance under 30 km. The same set of analyses considering the 7 events and the 3 return periods were performed for the bridge with damage model and for the elastic model. Figure 4 shows, as an example, the spectral compatibility procedure performed in the three directions N-S, E-W, Z for the return period of 975 years.

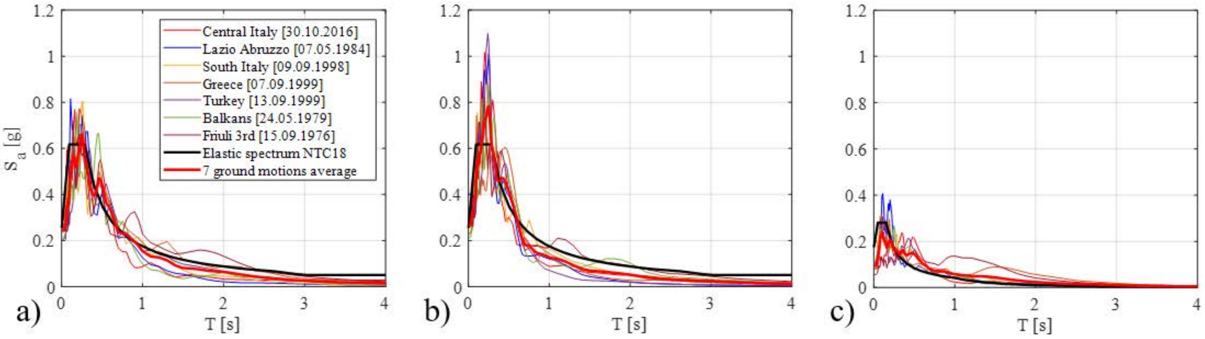

Figure 4: 975 years return period elastic spectra of the 7 selected seismic records, NTC18 elastic spectrum for the site (black), average spectrum of the 7 records set (red): N-S direction (a), E-W direction (b), Z direction (c).

The set was determined on the basis of the study of the historical seismicity of the site where the bridge is located by means of the Italian Macroseismic Database DBMI [15]. Then, the main focal mechanisms that affected the area by means of the Italian Accelerometric Archive ITACA [16] were determined, so as to obtain a set of parameters of interest useful in the following search for modern natural accelerograms with affine characteristics in the Engineering Strong Motion Database [17]. After applying the spectral compatibility procedure, the set to be employed in the time-history analyses was then composed. The Rayleigh damping method was employed to evaluate the structural damping, with a 5% damping ratio and considering the angular frequencies associated to the first two vibrational modes. Newmark implicit integration scheme with β and γ equal to 1/4 and 1/2, respectively, along with Newton-Raphson algorithm was adopted for the solution procedure.

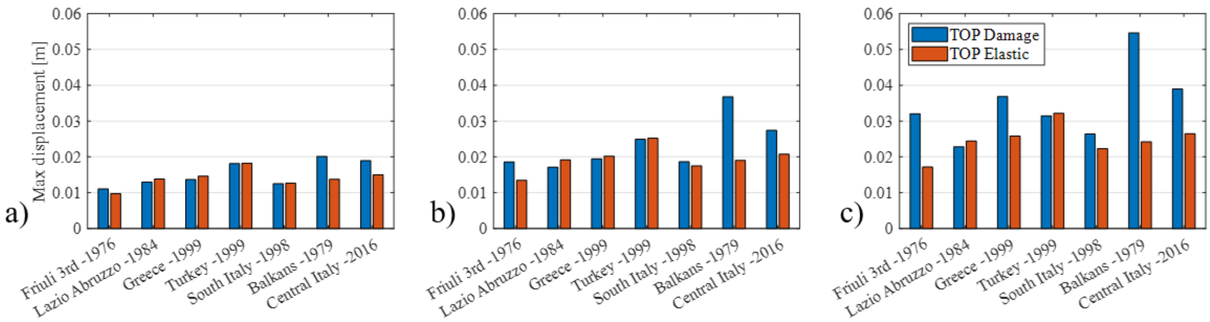

Figure 5: Maximum out-of-plane displacements (N-S) at the top point of the bridge for damage and elastic model: 201 year return period (a), 475 year return period (b), 975 year return period (c).

Figure 5 presents the results in terms of the out-of-plane (N-S) displacements of the TOP point for the 7 events, for the 3 return periods, for both the elastic and damage models of the bridge. In fact, the analyses show greater displacements along this direction: what can be observed is a change in trend going from the lower return period (201 years) to the higher return period (975 years). In the first case it can be noticed that the elastic model experiences maximum displacements in 4 of the 7 events, while in the second case the maximum displacement is reached by the damage model in most of the cases. This trend can be explained due to the phenomenon of dynamic amplification of the response in the damage model. In relation to the frequency content and intensity of the forcing action, the damaging structure experiences degradation during seismic action, which is associated to a change in the modal periods caused by the stiffness decay; therefore, new resonance conditions will be experienced compared to the elastic case. In Greece 1999 and South Italy 1988 events, this variation in the maximum out-of-plane response can be observed in the transition from the 201 year to 975 year return period; in contrast, the Turkey 1999 and Lazio-Abruzzo 1984 events always show at the different return periods a maximum response for the elastic model, so the damage model experiences a dynamic de-amplification phenomenon.

Focusing on the Turkey 1999 event with a return period of 975 years, the results obtained are shown in Figure 6; this is intended as an example of the procedure carried out in all the analyses. The largest displacements are found in the N-S direction, where in the first instants of analysis the model with damage and the elastic one show approximately identical oscillations in the response; at the explication of maximum damage level, around 4 seconds, the two responses start to differ significantly. Along the Z direction, it is interesting to note how the displacement oscillations go from an initial average value at the beginning of the analysis to a new value at the end of the analysis, that constitutes a final residual displacement. Figure 6 (b) and (d) show a clear correlation between the damage evolution in the structure and this variation in the displacement experienced in the Z direction: between 2.8 and 4 seconds, a strong increase of the damage index corresponds to the onset of residual Z displacement. Damage evolution was followed in all the analyses by monitoring the Global Damage Index (GDI) [6] and $D_{0.95}$ [11], both of which are useful indexes for assessing the average damage surveyed in the structure.

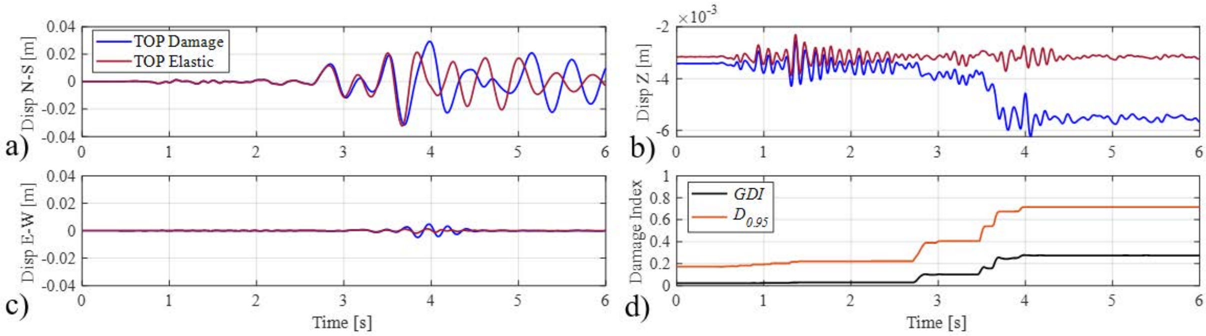

Figure 6: Displacements for damage and elastic model for N-S (a), Z (b) and E-W (c) directions, and damage indexes (d) for Turkey 1999 event with 975 year return period.

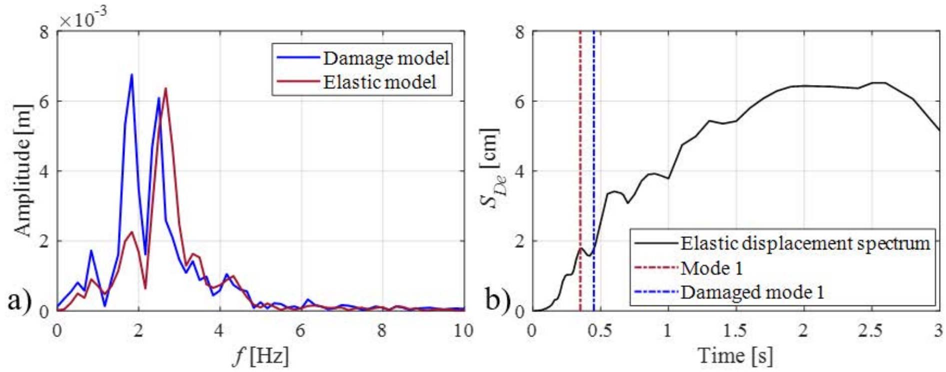

Figure 7: Fast Fourier Transform (FFT) of the out-of-plane displacement for Turkey 1999 record with 975 year return period (a); elastic displacement spectrum vs mode 1 period for elastic and damage model (b).

For the same event, the Fast Fourier Transform (FFT) is shown in Figure 7. These representations are useful to understand the strength and stiffness degradation phenomena occurring in the bridge, providing an assessment of the modified modal frequencies downstream of the damage occurrence. By comparing the first period of vibration of the elastic model and the damage one within the elastic displacement spectrum of the Turkey 1999 event, good agreement with the expected maximum response for the damage model can be observed. However, this procedure is exclusively based on the first mode of the structure and in the out-of-plane direction of the bridge only, without contemplating the influence of the longitudinal, bending and vertical modes, resulting in an excessively simplified assumption for an analysis performed with three seismic acceleration components.

The damage progression can be also monitored by means of color gradient maps. Figure 8 shows the damage pattern detected at the end of the analysis for the Turkey 1999 event with return period 975 years. What is observed is predominantly a low damage level throughout most of the structure, with a symmetrical distribution. Looking instead at the maximum damage reached, this creates a strongly non-symmetrical configuration, going to be concentrated in the West zone of the structure, in the region adjacent to the West lateral arch. All the crowns of the arches also show evidence of localized high damage. This distribution therefore shows an incidence of longitudinal components, which go to excite the higher modes of the structure.

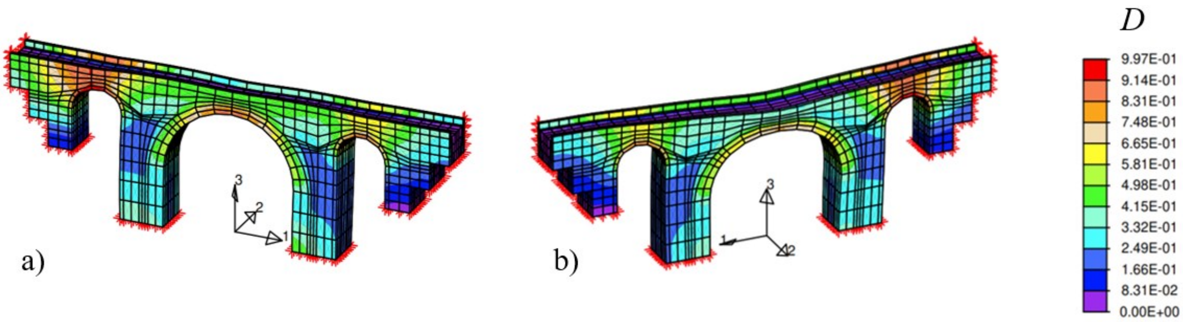

Figure 8: Damage patterns detected at the end of the analysis for Turkey 1999 event with 975 year return period: South face (a), North face (b).

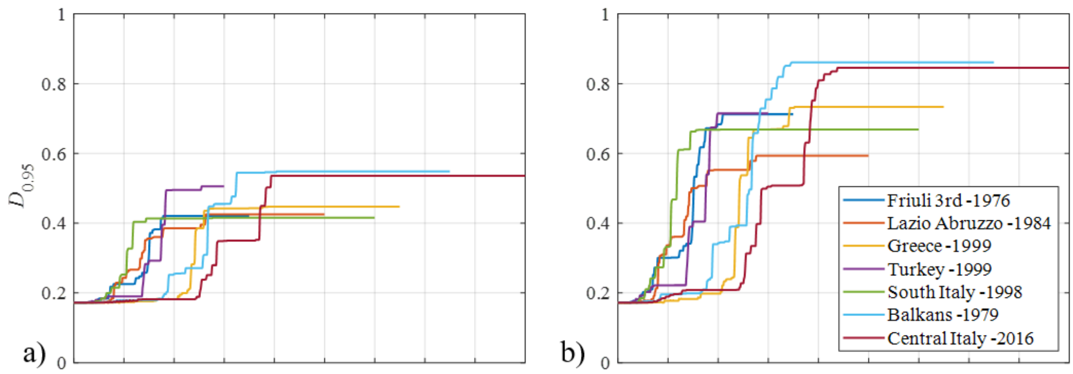

Figure 9: Damage index for the analyses on the damage model: 201 (a) and 975 (b) years return period.

Analyzing the summary graphs of $D_{0.95}$ for all the 7 analyses of the bridge damage model (Figure 9), for the lowest (201 years) and highest (975 years) return period, some observations can be done. As the return period, hence the intensity of the seismic action, increases, as expected, there are substantial increases in the maximum damage level reached at the end of the analysis, going from maximum index values of about 0.6 in the first graph to values even exceeding 0.8 in the second graph. With the same return period, it can be observed that the maximum damage value achieved, comparing the different events, is in perfect agreement with the trend in terms of maximum displacements (Figure 5).

With reference to the return period 975 years, the Lazio-Abruzzo 1984 event shows the lowest maximum damage of the set, to which the lowest maximum displacement value of the set is associated; in the same way, the Balkan 1979 event shows both the highest maximum damage and out-of-plane displacement. This damage index, measuring the average damage of the bridge during the analysis, turns out to be an essential marker of the health of the artifact, capable of synthetically showing the severity of the earthquake action on the construction, and the rapidity of development of the degrading phenomenon.

4 CONCLUSIONS

The vulnerability analysis of masonry constructions located in seismic zones constitutes a crucial topic nowadays. From this arises the need for an accurate description of the constitutive behavior of masonry material, which is a heterogeneous medium whose modeling is particularly complex. In this framework, the present work reported the study of a masonry arch bridge, currently employed in the roadway transportation system. A constitutive law with damage was employed to characterize masonry, which can faithfully capture the strain softening response of the material under cyclic earthquake actions. The artifact was investigated at different levels, starting from the linear field with modal analysis, moving on to estimate its capacity curve with pushover analysis, and then performing an assessment with natural accelerograms. The selection of earthquakes with different characteristics in frequency, M_w, focal mechanism, and epicentral distance was intended to subject the bridge to heterogeneous scenarios, but still strongly related to the previous seismic history of the site. The use of the three earthquake components allowed for modeling the action as closely as possible to site conditions under real earthquakes, while the choice to perform analyses at different return periods is related to the idea of examining different levels of increasing damage. The evolution of nonlinearities in the structure, and thus of damage, was followed by means of indices and damage maps, the former being essential tools for assessing the average damage recorded in the structure, and the latter for visually identifying the most vulnerable areas of the bridge. Therefore, performing this kind of analysis is the first step in outlining structural weaknesses, on which to operate, avoiding possible collapse mechanisms under seismic events. Further applications see more in-depth analyses of the behavior in near-fault earthquakes, where the Z component is decisively influential in the response.

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