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DAMAGE EVALUATION IN MASONRY INFILLED RC BUILDINGS FROM SHM DATA

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Abstract

In long-term structural health monitoring (SHM), one of the main challenges is to correlate an observed variation in dynamic properties of a structure (i.e., natural frequencies, mode shapes, etc.) to a certain damage level. This correlation is fundamental for a fast damage quantification trough SHM data that can trigger warnings for maintenance works, in the case of minor damage, or for immediate evacuation to protect the life of occupants in the case of a heavy damage. This paper focuses on seismic damage assessment, and adopts a methodology proposed by the authors to quantify the damage experienced in a building from SHM data. In detail, the study focuses on the damage evaluation of Reinforced Concrete (RC) buildings with masonry infill walls and adopts the variation of fundamental frequency as a damage intensity measure. Results from refined numerical simulations on a 2D infilled RC frame are reported for the structural and non-structural damage evaluation.

Keywords: Structural Health Monitoring, seismic damage evaluation, numerical simulations, natural frequency variation.

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1 INTRODUCTION

Existing buildings may experience slight to severe damage during earthquakes. A rapid identification of the structural health condition of existing structures is crucial for timely decision-making processes about interventions, minimizing service disruptions and avoiding economic and societal loss. The use of SHM data for the damage evaluation represents a suitable solution for a fast post-emergency management of critical buildings and existing assets. Several studies in the literature adopted SHM data for the model updating of structures and infrastructure for damage identification, localization and quantification. More recently, digital twins are also trained and employed for the damage evaluation of structures. All these methods are structure-dependent and requires a good knowledge level for the monitored structure.

As an alternative, threshold values for the variation of dynamic properties (i.e., natural frequencies, mode shapes, etc.) of buildings can be adopted to a fast seismic damage evaluation of existing assets. Recently, the authors investigated on the reliability of refined numerical simulations in providing correct estimations of dynamic properties variation as a function of damage through the model validation on results from output-only modal identification tests [1], [2]. A few studies experimentally investigated the effect of seismic damage on RC frames [3]–[5]. Others adopted numerical simulations and probabilistic analyses to quantify the effect of seismic damage on single structural components (i.e., columns) [6] or frames [7], [8]. However, the effect of non-structural components (i.e., masonry infill walls) on the variation of modal properties due to damage has never been properly considered.

The present study assesses through refined numerical simulations the effect of seismic damage on ductile RC frames with and without masonry infill walls. Non-linear time-history (NLTH) analyses are conducted on a 2D RC frame typical of existing buildings in the Mediterranean area. Modal analyses are performed before and after each NLTH analysis to monitor the variation of fundamental frequencies, herein considered as main damage intensity measure. Different ground motion intensities are considered to assess the variation of fundamental frequency for increasing damage level of structural and non-structural components.

2 CASE-STUDY FRAME

The selected case-study is a 2D two-storey single-bay frame that reproduces a perimetral frame of a real RC building heavily damaged during the L'Aquila earthquake (2009), and then demolished due to economic reasons [9]. The frame is 6.86 m high and 4.50 m long. The inter-storey height is 3.10 m. The square columns, 400x400 mm^2, are reinforced with 8 ϕ16 mm longitudinal bars and ϕ8 mm transverse reinforcement 250 mm spaced. The beams cross-section has a 500 mm height and 400 mm width and is reinforced with 6 ϕ16 mm and 4 ϕ16 mm bars at top and bottom, respectively. The infill walls are made by hollow clay bricks with four-side boundary conditions.

The frame is made with a medium quality concrete (i.e., 19 MPa) and the steel adopted for longitudinal and transverse reinforcement have a yielding strength of 535 MPa. The infill walls have a compressive strength of 2.59 MPa in the direction parallel to the holes and a compressive strength of 1.91 MPa in the orthogonal direction to the holes. Details about the case-study frame geometry are reported in Figure 1.

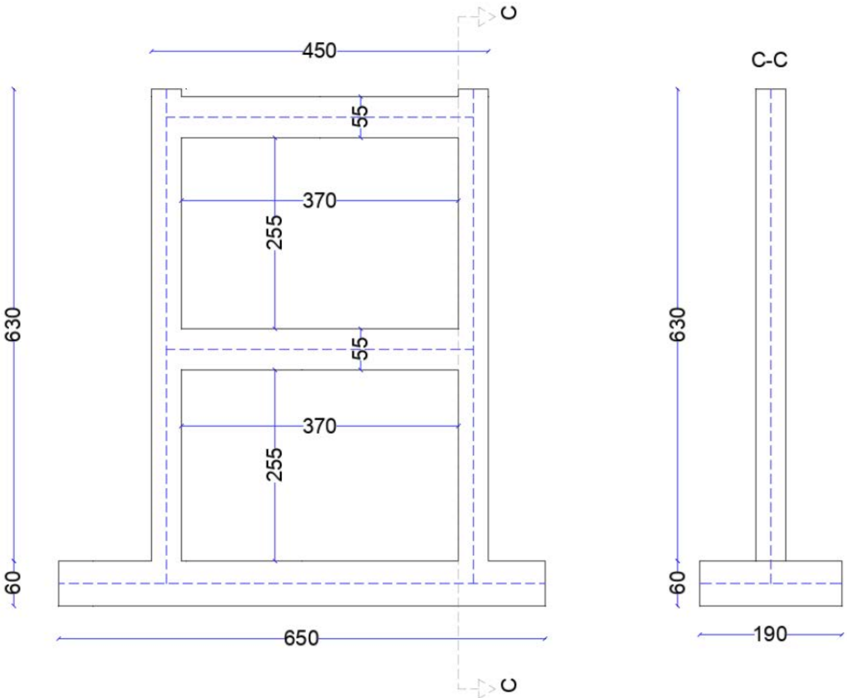


Figure 1: Case-study frame geometry, dimension in mm.

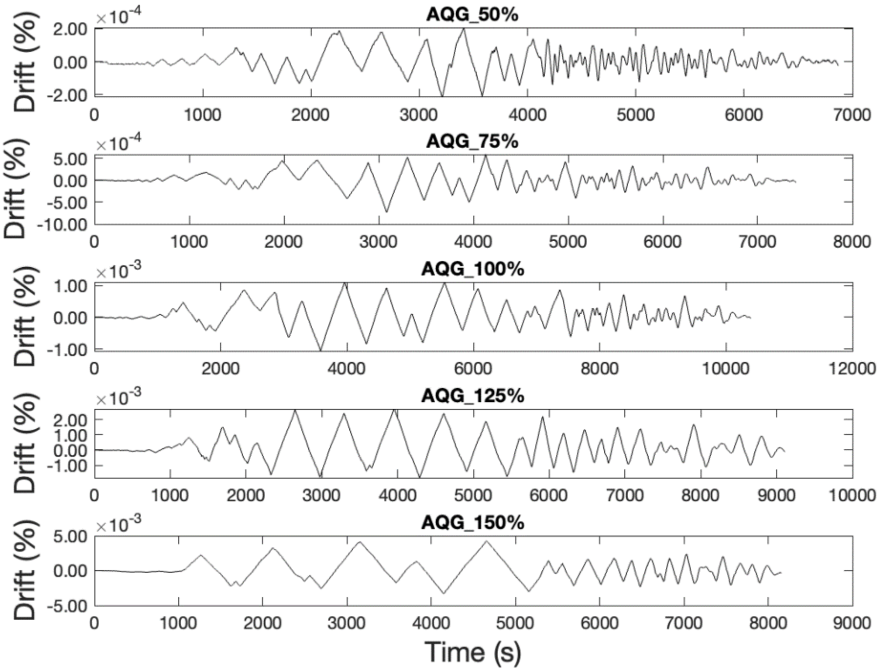
3 FINITE ELEMENT MODELLING

A refined non-linear FE model of the case-study frame is developed in *OpenSees* [10]. A schematic summary of the numerical model is reported in Figure 6 where the blue dots The *BeamWithHinges* command is used to build *forceBeamColumn* elements, which allow distributed plasticity to be spread also beyond the plastic hinge region. The solid cross-section of RC elements is uniformly discretised into fibres to closely represent small stress-strain variations. The effect of confinement of concrete mechanical behaviour has been neglected due to the low axial load ratio and high transverse reinforcement spacing. The concrete nonlinear behaviour is simulated with the *Concrete01* material, while the longitudinal steel reinforcement is modelled with the *OpenSees* uniaxial *Hysteretic* material. The parameters adopted for both stress-strain models are calibrated against experimental data reported in Del Vecchio et al. [9]. Beam-column joints are modelled as rotational springs with a *Pinching4* material, adopting the model proposed by De Risi et al. [11]. The nonlinear behavior of the infill panels is reproduced by adopting the three-strut model suggested by Chrysostomou et al. [12] in both directions, with struts acting in compression only. Truss elements with *Pinching4* material are used both for central and off-struts. The overall lateral performance of the infill wall is assessed following the multilinear model proposed by Panagiotakos and Fardis [13]. A mass of 5 tons is applied at the first storey level of the model, whereas a mass of 2.5 tons is applied on second storey to simulate the mass distribution of test specimens.

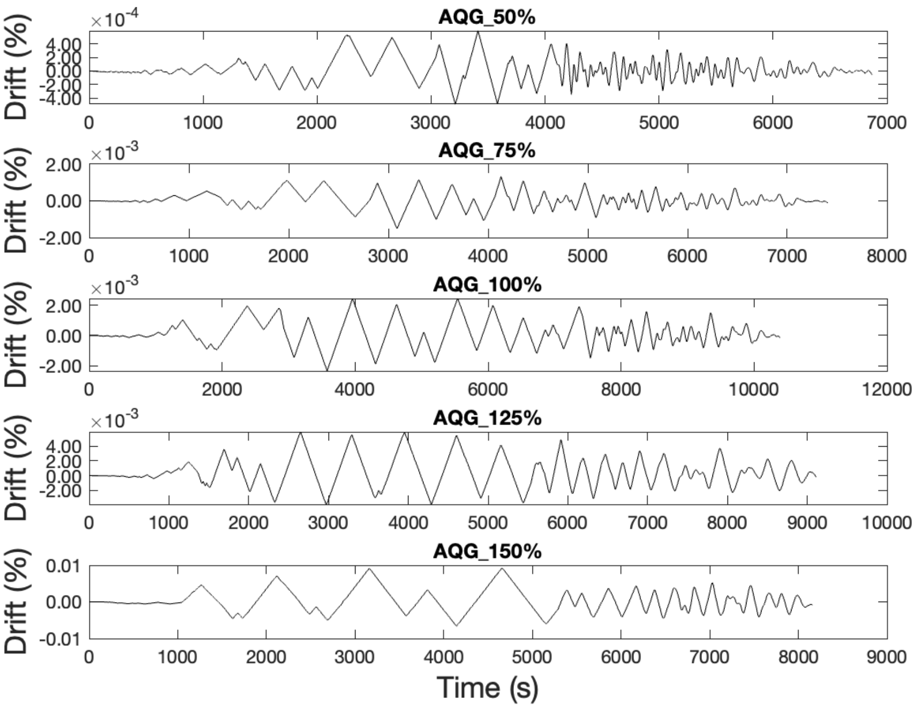
3.1 Non-linear time-history analysis

Nonlinear time-history analyses (NLTH) are carried out imposing as input data displacement histories at the two storey levels. The displacement histories have been derived by [14] performing non-linear dynamic analyses on the 3D infilled frame structure from which the case-study frame is extracted. The AQG record in the Est direction (peak ground acceleration

PGA=0.45 g) of the 2009 L'Aquila earthquake is used as input acceleration. Several increasing intensities are considered to simulated different damage levels on both structural and non-structural components (i.e., 50%, 75%, 100%, 125%, 150%), as shown in Figure 2.



a)



b)

Figure 2: Storey displacement histories for first floor (a) and second floor (b).

4 RESULTS AND DISCUSSION

NLTH analyses are performed on the case-study frame with and without infill walls to assess the effect of structural and non-structural damage on the overall fundamental frequency variation. Only the fundamental frequency related to the in-plane mode of the frame is considered. It should be noted that the same displacement histories are used for both the bare and the infilled frame. Conversely, the masses of the two models are different given the absence of the infill walls in the bare model.

4.1 Bare frame

The NLTH analyses results for the bare frame are summarized in Table 1 in terms of PGA, maximum interstorey drift ratio (IDR), fundamental frequency and frequency variation due to structural damage. The results show that ground shaking up to an intensity of 150% (PGA=0.669g, IDR=0.62%) induces in the frame a variation in fundamental frequency less than 10% with respect to the undamaged frequency of the bare frame. It is observed that the variation of frequency is almost constant even though the increasing ground motion intensity. From the numerical model, the longitudinal steel bars yielding is not reached in any loading sequence, whereas the cracking moment at the column base is achieved even for an earthquake intensity of 50%.

Earthquake intensity [%]	PGA [g]	IDR [%]	Frequency [Hz]	Frequency variation [%]
0	0.000	-	6.90	-
50	0.223	0.05	6.33	-8.3
75	0.335	0.10	6.30	-8.7
100	0.446	0.16	6.26	-9.3
125	0.558	0.40	6.25	-9.3
150	0.669	0.62	6.20	-10.1

Table 1. NLTH results for the bare frame.

4.2 Infilled frame

Similarly, NLTH analyses results for the infilled frame are summarized in Table 2. It is observed that, in comparison to the bare frame, the frequency of the undamaged infilled frame is about four times greater than the undamaged bare frame frequency.

Results show a significant variation of frequency (i.e., 26.3%) for a ground motion intensity of 75%, which follows the achievement of the infill panel cracking in the numerical model. A second important variation of natural frequency (i.e., 64.6%) is observed for a ground motion intensity of 150%, which follows the achievement of the infill maximum compression capacity in the FE model (i.e., infill crushing). Such results are in agreement with experimental results obtained in [2].

Earthquake intensity [%]	PGA [g]	IDR [%]	Frequency [Hz]	Frequency variation [%]
0	0.000	-	27.4	-
50	0.223	0.05	27.2	-0.7

75	0.335	0.10	20.2	-26.3
100	0.446	0.16	20.1	-26.6
125	0.558	0.40	19.3	-29.6
150	0.669	0.62	9.7	-64.6

Table 2: NLTH results for the infilled frame.

For the seismic damage level estimation, numerical results are compared with the seismic damage scale for existing RC frames with masonry infill walls proposed by Del Gaudio et al. [15]. According to the definition of damage levels for non-structural components, the infills cracking is classified as a DS1 while the infills crushing is classified as a DS3. Hence, for the case-study frame, a frequency variation of about 25% is associated to a non-structural damage DS1 and a frequency variation of about 65% is associated to a non-structural damage DS3. In terms of structural damage, the concrete cracking is considered as a DS.

For structural components, Del Gaudio et al. [15] classifies as DS1 – fine cracks in plaster over frame members - the first attainment of the cracking moment at the column end section, and as DS2 – cracks in columns - the first attainment of the yielding moment (or yielding chord rotation) in RC columns. Hence, for the case-study frame, a structural DS1 is reached for the considered loading sequences.

5 CONCLUSIONS

The present paper adopts a numerical approach for estimating the effect of seismic damage of ductile RC frames with and without masonry infill walls on their modal properties. Non-linear time-history (NLTH) analyses are conducted on a 2D RC frame typical of existing buildings in the Mediterranean area. Modal analyses are performed before and after each NLTH analysis to monitor the variation of fundamental frequencies at increasing ground motion intensities.

The study shows the potentiality of refined numerical simulations for the correlation between an observed variation in fundamental frequency of a monitored building and the occurred damage. The seismic damage scale herein adopted is the one proposed by Del Gaudio et al. 2017. More in detail, for the case-study frame, the non-structural components represent the most damaged parts of the frame, significantly affecting the overall fundamental frequency with a variation of 25% for a DS1 and of 65% for a DS3 with respect to the undamaged frequency. This approach goes towards the definition of correlation matrices between fundamental frequency variation and seismic damage levels for a fast damage assessment of existing assets.

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