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VALIDATION OF REFINED NUMERICAL MODELING FOR EXISTING RC BUILDINGS: COMPARISON BETWEEN PREDICTED AND OBSERVED EARTHQUAKE DAMAGE

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Keywords: Numerical modeling, earthquake damage, non-linear behavior, RC structures, infill wall, beam-column joints.

Abstract. *A reliable estimation of the seismic performances of existing reinforced concrete building is of paramount importance to design proper retrofit solutions. Number of refined numerical models are nowadays available, nevertheless predicting the seismic performance and the earthquake damage at component level is still a challenging task. Marked nonlinear phenomena, strength and stiffness degradation and pinching affect the cyclic behavior of structural members designed with reinforcement details non-conforming with current seismic codes. This, along with the interaction between the bare frame and the stiff infills, makes complex the reproduction of the global structural behavior. This study focuses on a refined modeling procedure properly developed for existing RC frames. The modeling assumptions, the hysteresis assigned to the different building components and the modeling strategy accounting for the infill contribution are presented and discussed. The model validation at component and building level related to case-study buildings damaged by the L'Aquila (2009) earthquake is presented. A component-by-component comparison between predicted and observed damage is shown. The proposed numerical model and the in-depth discussion on the earthquake damage are useful to identify the building weaknesses, estimate the repair costs and design proper retrofit solutions.*

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1 INTRODUCTION

Recent seismic events showed the significant damage to existing RC buildings and the amount of economic and social resources involved in the reconstruction process [1–3]. Scientific studies and damage observations in the aftermath of the L'Aquila 2009 earthquake [4,5] outlined that infill wall and partitions are the most damaged nonstructural components (see Figure 1a). This is because of the interaction with the surrounding RC frames and to the high stiffness of hollow clay brick systems typical of the Mediterranean. The repair costs concerning the repair/substitution of damaged infill, finishes and services (i.e. plumbing and electrical system, commonly incorporated in the infill or partitions) represent the majority of the total building repair cost. Thus, reliable earthquake loss assessment procedures necessarily have to account for the contribution and the damage of infill/partition walls [6,7]. Furthermore, the presence of infill walls significantly increase the building stiffness increasing the seismic demand transmitted to the structural members and the occurrence of shear failures (see Figure 1b,c). This makes the modeling of such components of paramount importance to have reliable estimation of the earthquake damage, of the seismic capacity and to design proper retrofit solutions.

(a) (b) (c)

Figure 1: L'Aquila 2009 earthquake damage: severe damage to exterior infill (a); cracking of a corner beam-column joint (b); damage to stairs and short columns (c).

The advanced numerical models proposed in the recent years improved accuracy in reproducing the nonlinear behavior of RC structural systems [8,9] and the contribution of infill walls. New mechanical models and solution algorithms have been developed, and a great effort has been made to combine them with the state-of-the-art knowledge in refined tools (e.g., [10,11]) suitable for the use in the practical applications. However, they requires continuous calibrations and validations both at subassembly and at system level. Recently, several lumped plasticity models have been proposed to reproduce the nonlinear behavior of poorly detailed beam-column joints [12] and infill walls [13]. They were widely validated on experimental tests on subassemblies or entire structural systems [14,15].

This paper deals with the application of a refined modeling strategy in a nonlinear FEM environment to case study RC buildings experiencing significant damage to non-structural components. The building characteristic, the modeling procedures and the earthquake damage are described in detail. Particular attention is reserved to the damage quantification and modeling of infills and RC joints. A component-by-component comparison between predicted and observed damage is shown. A useful correspondence between engineering demand parameters (EDPs), available damage states classifications and damage scale commonly adopted in the inspections in the immediate aftermath of the earthquakes is proposed.

2 CASE STUDY BUILDINGS

The reconstruction process after L'Aquila earthquake (2009) has been a unique occasion to collect reconstruction costs data at large scale. A detailed description of the reconstruction policy, the regulation and an overview of the data related to the reconstruction of 5,775 residential buildings damaged by the L'Aquila earthquake is reported in Di Ludovico et al. [2,3]. To obtain the public founding for the reconstruction of the damaged buildings, documentation was required to illustrate the damage and the design of repair and strengthening interventions. A team was set up to oversee these projects and establish the technical and economic feasibility of the intervention. All the documents were collected in a big database containing detailed information on the buildings geometry, material properties, reinforcement details, earthquake damage and technical and economic documents submitted for funding application. The existing buildings described in this database are of particular interest because they have material properties and reinforcement details typical of Mediterranean building stock and very difficult to reproduce. Furthermore, the damage detected in the aftermath of the L'Aquila earthquake (2009) reflects the building weaknesses typical of the existing buildings. The buildings were classified using the AeDES [16] classification which directly reflect the damage severity.

In this study, two case study buildings were selected from the L'Aquila database and analyzed in detail. RC buildings with moment resisting frames and stiff infills (i.e. hollow brick wall) have been investigated as representative of residential buildings in the Mediterranean area. In order to reduce the uncertainty in the input motion, which may strongly affect the structural response and, in turn, the resulting damage, the case-study buildings were selected in the L'Aquila municipality and very close to the epicenter. The two buildings are located close to the accelerometric stations AQU and AQK (maximum distance about 2.5 km, see Figure 2).

Building 1 (B1) is a four storey building (plus basement), built in the 1972-1981 period and rated in the B-C class due to non-structural damage only and low repair cost. Building 2 (B2) is a three storey building, built in the 1982-1991 period and rated in the E-B class.

The case study buildings experienced different levels of non-structural damage and only limited damage to structural components (beam-column joints). **Error! Reference source not found.-Error! Reference source not found.** show the building elevation views and the main damage detected on structural and non-structural components due to the 2009 L'Aquila earthquake. The location of the damage is also reported. The building B1 (class B-C) experienced in general slight damage (with the exception of very heavy damage for a few panels) to infill, partitions and to the floor finishes (see **Error! Reference source not found.**). The building B2 (class E-B) experienced significant damage to non-structural components such as infill, partitions and floor finishes (see **Error! Reference source not found.**).

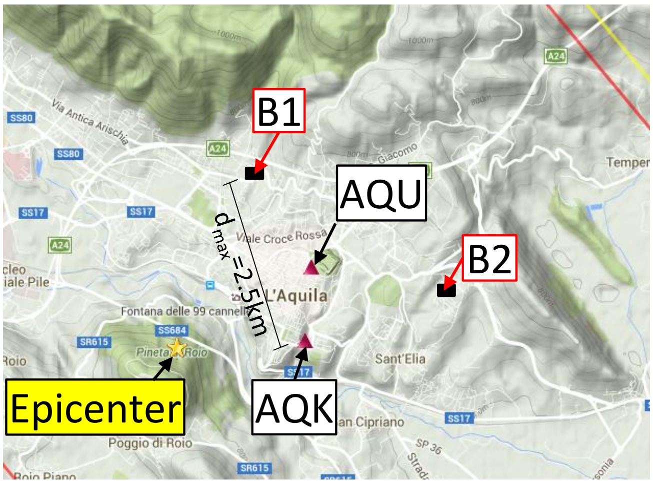


Figure 2: Building location and accelerometric stations of the L'Aquila earthquake (2009).

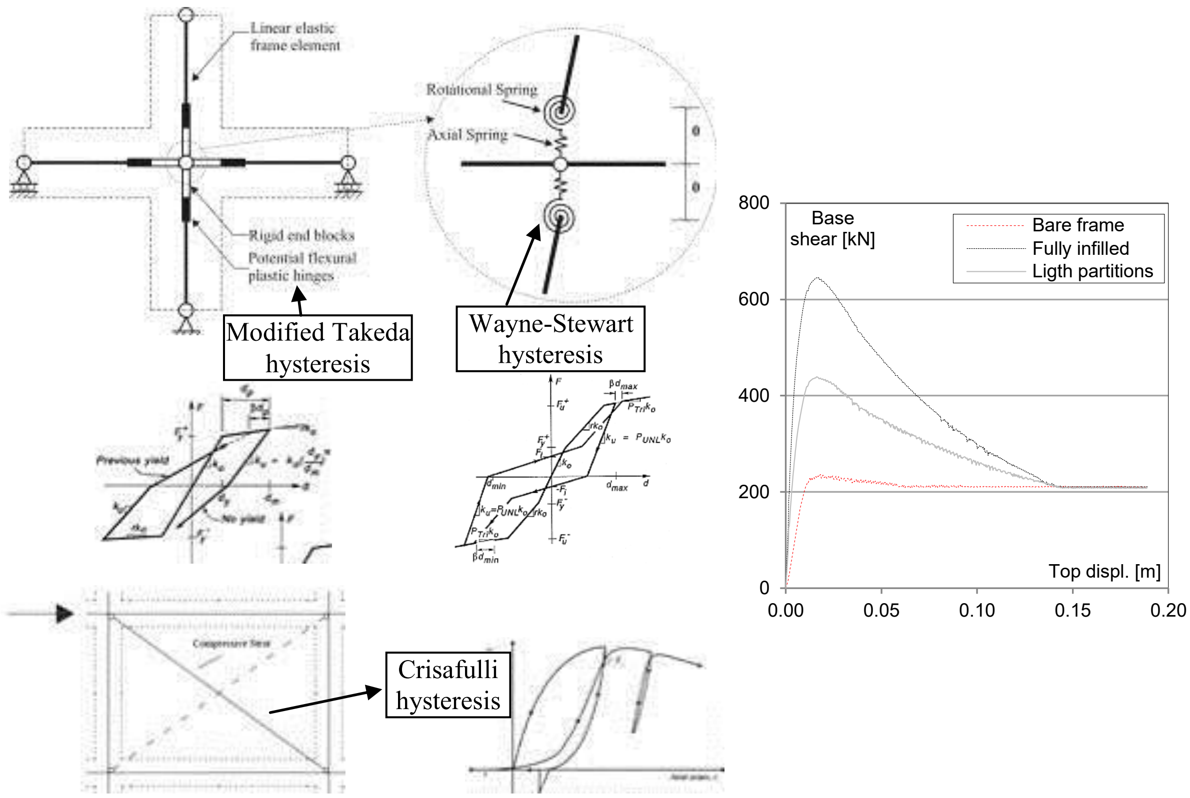
All the selected buildings rely on RC moment resisting frames as primary structural system and have in plan and elevation structural regularity (see Figure 6a,c and Figure 7a,c), in order to avoid particular cases where the reconstruction costs could have been strongly affected by irregularity in the structural response. The reinforcement details of beams and columns are summarized in Table 1.

				Direction X						Direction Y					
				Columns			Beams			Columns			Beams		
ID	f_{cm} [MPa]	f_{ym} [MPa]	Storey	h (b) [mm]	$A_{s,t}$ $=A_{s,b}$	Stirr. d (sp.) [mm]	h (b) [mm]	$A_{s,t}$ $(A_{s,b})$	Stirr. d (sp.) [mm]]	h (b) [mm]	$A_{s,t}$ $=A_{s,b}$	Stirr. d (sp.) [mm]	h (b) [mm]	$A_{s,t}$ $(A_{s,b})$	Stirr. d (sp.) [mm]
B1	20	380	All	300 (500)	3ϕ16	ϕ8 (200)	500 (300)	5ϕ16 (3ϕ16)	ϕ8 (200)	500 (300)	2ϕ16	ϕ8 (200)	500 (300)	5ϕ16 (3ϕ16)	ϕ8 (200)
B2	29	516	All	300 (500)	3ϕ16	ϕ6 (100)	500 (300)	5ϕ16 (4ϕ16)	ϕ6 (60)	500 (300)	2ϕ16	ϕ6 (100)	500 (300)	5ϕ16 (4ϕ16)	ϕ6 (60)

Table 1: Material properties and reinforcement details of the case study buildings.

3 NUMERICAL MODELING

A two dimensional refined numerical model developed in the finite element method (FEM) software Ruaumoko [11] has been adopted to assess the seismic performances of case study buildings. In order to reproduce the building performances by using a single 2D frame model, a proper distribution of the floor masses, considering the number of frames in each direction and the different stiffness, is needed. A lumped plasticity approach, concentrating RC member nonlinearities in critical members such as portion ends of beam and column, beam-column joints and stiff infill walls has been adopted. Number of literature studies [17,18] and inspections in the aftermath of recent seismic events demonstrated the high vulnerability of poorly detailed beam-column joints. Numerical studies [9,17] pointed out that modeling the nonlinear behavior of beam-column joints is of paramount importance to properly assess the building seismic performances. Stiff infill walls and partition have been found to significantly affect the building global and local response [14,15]. To account for these criticism in the seismic performance of existing RC frames typical of the Mediterranean area, proper capacity models and refined hysteretic rules have been proposed for beam-column joints [12], non-conforming RC members (modified Takeda, [19] and clay brick in-fills [13]. For this purpose the influence of joint response on rotational capacity of faming members (with not negligible effects on the interstory drift and frame deformability) has been considered including rotational springs (see Figure 3a). Plastic hinges and lumped springs have been characterized basing on member geometries, reinforcement details and material properties available in structural drawings and material properties characterization tests performed by the practitioners engaged for funding applications. The material properties, the member cross-sections and the reinforcement details used for the plastic hinge characterization are summarized in Table 1. Beams and columns were modelled by mono-dimensional elastic elements with inelastic behavior concentrated at the edges in plastic hinge regions (Giberson model, Figure 3a) and defined by appropriate stiffness degrading moment-curvature hysteresis rules available in the library of Ruaumoko to account for shear strength degradation and poor transverse confinement.



(a) (b)

Figure 3: Adopted numerical model after Galli [15] (a); and typical capacity curve of building frames (b);

The modified-Takeda hysteresis model [19] was adopted for beam and column elements. To account for the axial load variation in the column members the yield surface was defined based on the moment-axial load strength domain. The cyclic behavior of the joint rotational spring has been modeled using the Wayne-Stewart [11] hysteresis model (Figure 3a) able to describe the stiffness degrading characteristics of a beam-column joint with the peculiar expected pinching effects typical of cracked joints [12]. The cyclic behavior of the infill panel was been modelled adopting the hysteresis rule proposed by Crisafulli [13] to simulate the axial response. Infill mechanical properties have been quantified according to available literature formulations [20] calibrated on experimental tests on infill wall typical of the Italian constructions. Diagonal strut connects beam and column ends in correspondence of the plastic hinges by means of stiff elements able to transmit only compression actions. This in order to account for the infill action on RC member deformability. The proposed numerical models have been widely validate at both at subassembly and frame levels [15,20] under static and dynamic actions.

Pushover analyses were carried out to investigate the damage and collapse mechanism and the differences in terms of initial stiffness between bare-frame, fully infilled frame and interior frame with partitions (see Figure 3b). Capacity curves typical of the case study buildings show a collapse mechanism characterized by joint shear failure involving the first storey members. The infill wall and interior partition significantly affect the frame response in terms of strength, stiffness and collapse mechanism. This need to be properly accounted in the mass distribution on each frame to be representative of the overall building response.

The numerical model of the exterior frame was then used for NLTH analyses. The gravity loads assigned to the reference frame was computed considering the frame tributary area. However, the seismic mass assigned at each floor was computed to reproduce the overall building response in the reference direction. In particular, the mass was distributed between bare frame, infilled frame and interior frame with light partitions as a function of the frame lateral stiffness (see Figure 3b) and number of frames in each direction [7]. This procedure allows to account for the higher stiffness of the fully infilled exterior RC frame, resulting in higher lateral forces transmitted to the structural member and the foundation. The adequacy of the developed numerical model is investigated by comparing the predicted damage with that observed in the aftermath of the earthquake for the reference case studies, as discussed in the further section.

The gravity loads assigned to the reference frame has been computed considering the frame tributary area. The numerical model of the exterior frame has been used for NLTH analysis. However, the seismic mass assigned at each floor has been computed to reproduce the overall building response in the reference direction. In particular, the mass has been distributed between bare frame, infilled frame and interior frame with light partitions as function of the frame lateral stiffness. The number of frames available in the structural system is known from structural and architectural drawings and confirmed by in-situ inspection carried by engaged practitioners for funding applications. With these assumptions the numerical model is able to reproduce with reasonable accuracy the building lateral response in each of the two directions. A further validation, comparing the predicted and observed damage is reported below.

L'Aquila 2009 earthquake's mainshock, Magnitude (Mw) of 6.3 was characterized by a normal fault mechanism 8.8km depth and epicenter about 6 km far from the city of L'Aquila. Earthquake ground shaking was considered in the form of acceleration histories recorded at AQU and AQK stations acquired on firm soil, i.e. soil class B according to Eurocode classification [21]. The accelerometric signals have a PGA ranging between 0.25-0.35g and were extracted from the Italian Accelerometric Archive [22]. Horizontal components of the chosen records were rotated in the orientation in plan of lateral resisting frames of the buildings. Geometrical formulations commonly adopted to identify the two components in near fault ground motions [23] were adopted. This allow to account for the pulse effects of the ground shaking at specific sites (AQK, as outlined by [24].

4 PREDICTED VS. OBSERVED DAMAGE

The simulations pointed out that none of the buildings reached the collapse, but they experienced significant structural and non-structural damage. The largest drift are concentrated at the first floors and significant non-structural damage to infill and partitions were detected.

A direct comparison between observed and predicted damage is proposed in order to assess the accuracy of the adopted numerical model to reproduce the earthquake response. The actual earthquake damage were classified using the AeDES [16] classification. It reports the damage grade (Slight, Medium-Severe, Very Heavy) and the damage extent on the entire building (<1/3, 1/3-2/3, >2/3) of structural and non-structural components (i.e. Vertical structures, Floors, Stairs, Roof, Infill-Partitions). The detected damage are reported on the building front view in both the X and Y direction, based on the results of the AeDES form and the damage observation available in the damage report and pictures.

To compare the observed earthquake damage and the damage predicted by means of the numerical model, a correspondence between Engineering Demand Parameters (EDPs), resulting from NLTH analysis, and damage states (DS) needs to be adopted. In the following paragraphs, the correspondence between EDPs and damage states for infill walls and beam-column joints is reported basing on available literature studies and the observations used to develop the AeDES damage classification.

The detected damage, the EDPs estimations and the nonlinear response of the structural and non-structural components for the two case study buildings are reported in Figure 6, Figure 7 and Figure 8.

4.1 Correspondence between damage levels and EDPs for infill walls

The AeDES damage classification [16] is based on the EMS98 scale [25] and it identifies the damage to infill and partitions with three damage grades: D1, Small diagonal cracks (⩽ 1 mm), small separation of the masonry panels from the structure; D2-D3, diagonal cracks or displacements of few mm, visible crushing at the infills corners; D4-D5, total collapse of infill panels. An example of the different damage grades is reported in top of Figure 5.

Even though different capacity models and experimental tests are available in literature, few works proposed a correspondence of the damage states and the EDPs. With reference to infill walls, a first correspondence between the damages and EDP expressed as function of the axial strain in the diagonal compressive strut, was proposed by Magnes and Pampanin [14] as function of the axial deformation in the equivalent strut, ε'_w, commonly used to model the infill action. In particular, no damage or minor cracking (DS1) $\varepsilon'_w < 0.002$; extensive damage (DS2) $0.002 \leq \varepsilon'_w < 0.005$; severe damage/incipient collapse (DS3) $0.005 \leq \varepsilon'_w$.

Sassun et al. (2016)-All type of Infill										Corresponding AeDES Damage level
DS	Description	IDR (%)				ε_{strut} (%)				
[-]	[-]	median	β	16th perc.	84th perc.	median	β	16th perc.	84° perc.	[-]
DS1	Ligth cracking (w<1mm)	0.18	0.52	0.11	0.30	0.08	0.51	0.05	0.13	D1
DS2	Extensive diagonal cracking (1<w<2mm)	0.46	0.54	0.27	0.79	0.22	0.52	0.13	0.37	D2-D3
DS3	Brick unit crushing or spalling, sliding joints, local collapse	1.05	0.40	0.70	1.57	0.51	0.40	0.34	0.76	D4-D5
DS4	Near collapse	1.88	0.38	1.29	2.75	0.89	0.37	0.61	1.29	

Note: the percentile underlined represent the limits assigned to the AeDES damage levels

Table 2: Correspondence between available damage states, the relevant EDPs and the AeDES damage levels

Furthermore they introduced a simple relationship which allows to estimate the interstorey drift knowing the strut axial deformation and the panel geometric aspect ratio (L/H).

$$\delta = \frac{L}{H} - \sqrt{(1-\varepsilon_w)^2\left(1+\left(\frac{L}{H}\right)^2\right)-1}$$

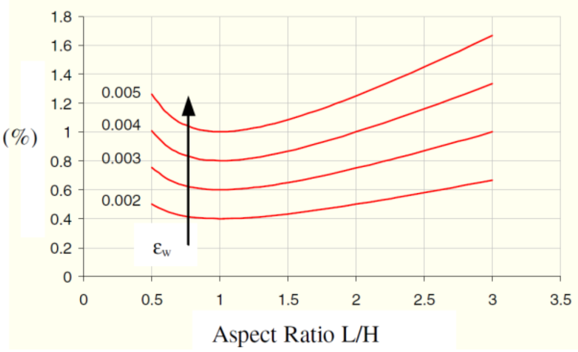


Figure 4: Relationship between strut axial strain, interstorey drift and aspect ratio L/H [14].

Recently, Cardone et al. [26] and Sassun et al. [27], proposed fragility functions for the infill and partitions based on a wide database of experimental tests for four the damage states. They correlate the probability of achieving a specific damage with the experimental interstorey drift or the the axial deformation in the equivalent strut, ε'_w, estimated with a numerical model. The results are summarized in Table 2 which also report the correspondence between the DS assumed by Cardone et al. [26] and Sassun et al. [27] and the damage grades of the AeDES form. This correspondence is proposed based on the damage description of the damage grades or damage states as summarized in Figure 5.

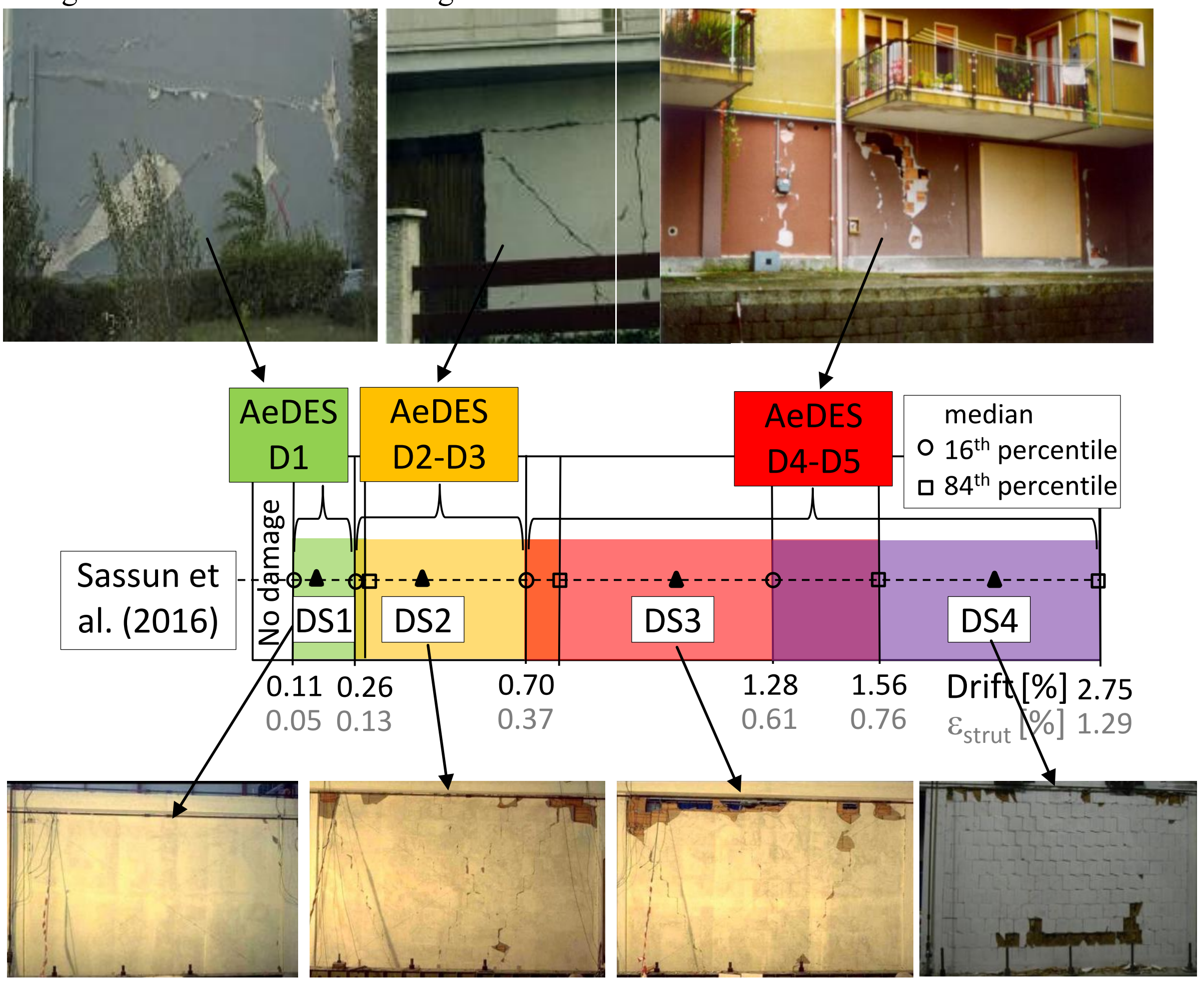


Figure 5: Correspondence between available damage classifications for clay brick infill (after [16,27]).

The interstorey drift predicted using the formulation proposed by Magnes and Pampanin [14] at the axial strain limits proposed by Sassun et al. (2016) well matches the intersotrey drift limits proposed by Sassun at al. (2016) and based on experimental observations. This confirm the accuracy of the formulation proposed by Magnes and Pampanin [14] which also account for the geometry of the infill.

It is worth mentioning that the damage grades proposed by the AeDES form include also the effects of the out-of-plane actions on the infill and the corresponding damage. By contrast, reliable and accurate damage descriptions and the corresponding EDPs due to the out-of-plane actions are currently not available due to the lack an adequate number of experimental tests. Thus, the damage predicted by using numerical models or available literature damage classification could underestimate the actual earthquake damage.

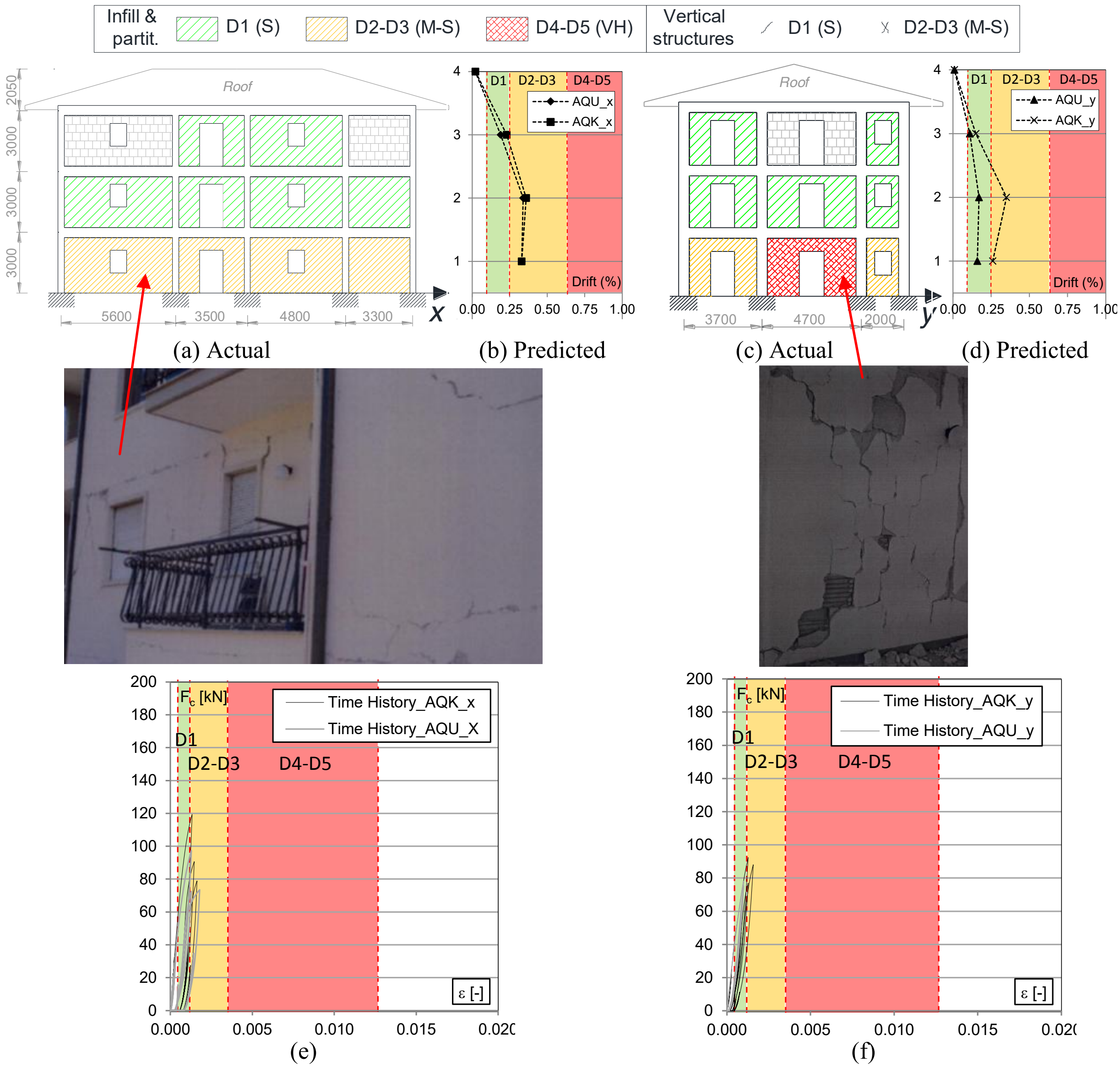


Figure 6: Building B1 (B-C) comparison between observed and predicted earthquake damage: actual damage in the X direction (a); predicted drift in the X direction (b); actual damage in the Y direction (c); predicted drift in the Y direction (d); predicted damage to exterior infill in the X and Y direction (e) and (f), respectively;

The detected earthquake damage on B1 in the X direction, see Figure 6a, were medium-severe (D2-D3) on the ground floor infill and slight (D1) on the upper floors. This well matches with the damage associated to the drift response, estimated by means of refined NLTHs, see Figure 6b, and the local damage achieved by the equivalent strut representative of the infill action, see Figure 6e.

In the Y direction, the building B1 exhibited medium-severe damage at the ground floor infills, see Figure 6c, matching with the predicted damage in terms of interstorey drift, see Figure 6d. It is worth mentioning that in-situ inspection reported a very high damage (D4-D5) to one of the ground floor infill (see Figure 6c) due to out of plane actions. Actually, this cannot be considered with available capacity models which predicted a D2-D3, medium-severe damage, see Figure 6d,f.

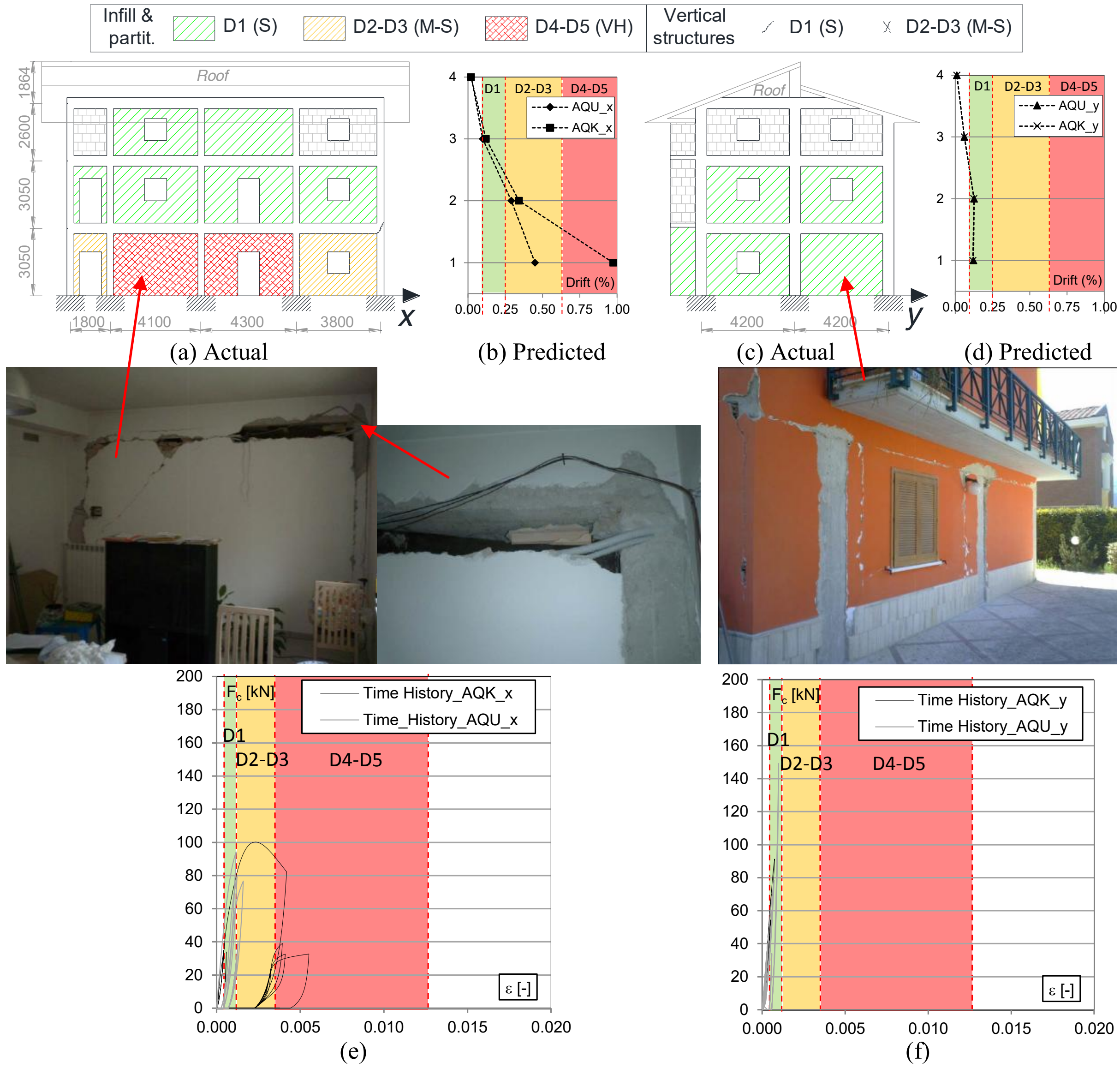


Figure 7: Building B2 (E-B) comparison between observed and predicted earthquake damage: actual damage in the X direction (a); predicted drift in the X direction (b); actual damage in the Y direction (c); predicted drift in the Y direction (d); predicted damage to exterior infill in the X and Y direction (e) and (f), respectively.

Figure 7a shows the extensive damage to ground floor infill. Medium-Severe (D2-D3) and very high (D4-D5) damage were detected on the B2 in the direction X. This is well predicted both by means of the proposed intertorey drift-damage correlation (Figure 7b) and in terms of stress-strain response on the single infill panels, see Figure 7e. Figure 7c shows the slight damage (D1) on ground and first floor infill of the B2 in the direction Y. Also in this case the predicted earthquake response match with the observed damage (see Figure 7d, f).

4.2 Correspondence between damage levels and EDPs for beam-column joints

A proposal to correlate the earthquake damage and the EDPs for poorly detailed beam-column joints was proposed by Magenes and Pampanin [14]. In particular, damage to poorly detailed beam-column joints can be expressed as function of the joint shear distortion, γ : first diagonal cracking (DS1) $0.0002 \leq \gamma < 0.005$; extensive damage (DS2) $0.005 \leq \gamma < 0.01$; severe damage/incipient collapse (DS3) $0.01 \leq \gamma$. The comparison between the proposed damage states and the nonlinear response of the corner joint at the first floor of the Building B2 that experienced slight shear cracking is reported in Figure 8.

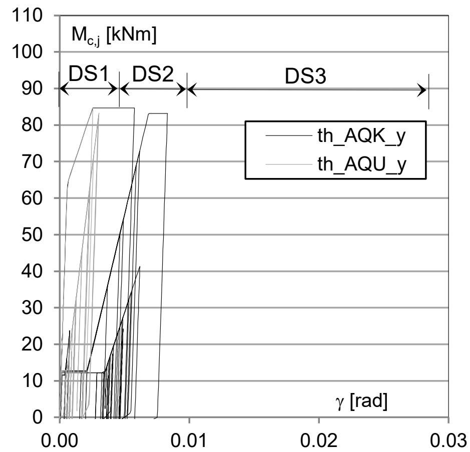


Figure 8: Building B2 (E-B) comparison between predicted nonlinear time-history response and damage states: for the corner joint at the first level in the X direction.

The comparison outlined a significant difference between the response due to the AQU and AQK record. This is because of the pulse effects related to the AQK record [24], which may lead to increase the earthquake demand on the structural system. On average, a slight to medium damage (DS1-DS2) to the joint panel is predicted, which matches with the in-situ observation.

5 CONCLUSIONS

The paper presents a refined numerical model developed in a FEM environment able to reproduce the nonlinear behavior of existing RC buildings both at global and component level. This allow the use of the proposed model in refined loss-assessment methodologies. The comparison between observed and predicted damage at global and component level of two case study buildings outlines a reasonable accuracy of the numerical models. In particular:

- The proposed numerical model is a reliable tool to reproduce the nonlinear dynamic response of existing RC buildings subjected to recorded acceleration histories. The capabilities of the model to reproduce the nonlinear behavior of infill wall with significant strength and stiffness degradation allow to reproduce severe damage states and the interaction with the RC frame.

- Two case study buildings with different damage levels on the infills and beam-column joints were used for the model validation. The comparison shows the good match between observed and predicted damage to infill walls at different levels experiencing slight, severe and very heavy earthquake damage.

- The proposed correspondence between AeDES damage grades and the EDPs allows to predict the damage to infill and partition using a floor-by-floor estimation of the interstorey drift with reasonable accuracy.

- A significant gap between actual and predicted damage can be observed for the infill panel experiencing out-of-plane damage due to the bidirectional effect of the ground shaking. This phenomena significantly affects the damage to drift-sensitive non-structural components and a further research effort is needed to develop detailed relationships between damage states and EDPs.

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