SEISMIC PERFORMANCE ASSESSMENT OF AUTOCLAVED AERATED CONCRETE (AAC) MASONRY BUILDINGS

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ABSTRACT :

Autoclaved aerated concrete (AAC) blocks are diffusely used worldwide as a construction material both for infill panels and load-bearing walls, because of their superior properties of fire resistance and thermal insulation.
The application and use of such technology in seismic prone areas, however, still requires further verification of the expected structural performance.
The very low weight of this material and its high deformability (low value of Young modulus in compression) tends to reduce inertia forces on the building induced by the seismic motion. On the other hand the masonry compressive strength of AAC, although its variability is extremely limited, is rather low compared to other traditional masonry types.
In order to assess the seismic behaviour of entire AAC masonry buildings, a calibration of the nonlinear macro-element included in the TREMURI analysis program was carried out based on the experimental cyclic response of masonry piers observed in tests performed at the EUCENTRE and University of Pavia laboratories.
Hence, several nonlinear static and dynamic analyses were carried out on complete building models, considering different structural configurations representative of typical Italian masonry building typologies.
The results obtained from adaptive pushover analyses (Actual Displacement-Based Adaptive Pushover) of the different prototypes have been compared with the results of incremental dynamic analyses.

KEYWORDS: AAC, masonry, seismic performance, non linear analyses, adaptive pushover, macro-element

1. INTRODUCTION

The use of autoclaved aerated concrete (AAC) load-bearing elements is diffusely used worldwide and they possess interesting material properties regarding earthquake engineering. Indeed their high deformability allied to their low weight reduces the inertia forces of these vertical elements and, in addiction to their non-combustibility and fire-resisting nature of AAC (earthquakes are commonly associated with fires), they may be an alternative to reinforced concrete frame structures. On the other hand masonry structures are commonly associated with poor seismic performance as observed in past earthquakes. This negative perception is caused mainly by many non-engineered masonry structures, mostly stone masonry houses which, if not properly designed and/or strengthened regarding seismic provisions, will not behave satisfactory under seismic excitations.
On the contrary, modern approaches to masonry constructions regarding seismic detailing with convenient conception or innovative materials and solutions, may lead to safer and economical constructions especially concerning small constructions.
Therefore a complete methodological approach to the seismic performance assessment of unreinforced AAC masonry buildings is presented on this work, enhancing the possibility to use nonlinear static procedures in the reproduction of the dynamic behaviour of AAC masonry buildings.

2. DESCRIPTION AND CALIBRATION OF THE NUMERICAL MODEL

2.1. The nonlinear macro-element model

The nonlinear macro-element model used in this work is based on the original macro-element formulation proposed by Gambarotta and Lagomarsino (1996) and implemented in the TREMURI program (Galasco *et al.*, 2006). Indeed the macro-element model, representative of a whole masonry panel, reproduces, on the basis of mechanical assumptions and with a limited number of degrees of freedom, the two main in-plane masonry failure modes: bending-rocking; shear-sliding (with friction). The algorithms embedded in the software may be consulted in several works performed on this domain (e.g. Penna, 2002; Lagomarsino and Penna, 2003; Galasco *et al.*, 2004; Penna *et al.*, 2004; Lagomarsino *et al.*, 2007).

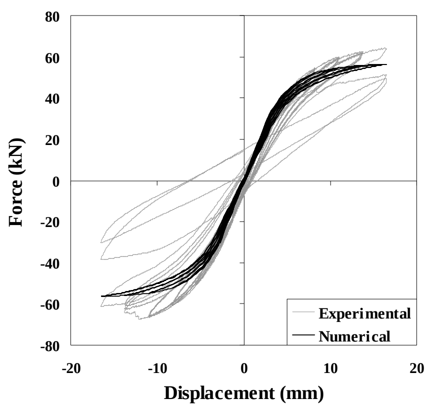
2.2. Calibration of the model parameters

In order to calibrate the model to reproduce correctly the in-plane behaviour of AAC masonry panels, the results obtained during the experimental test campaign performed on AAC wallettes and panels at EUCENTRE and University of Pavia laboratories (Costa, 2007; Penna *et al.*, 2007; Penna *et al.*, 2008) were used.
Therefore the initial trial values used on the calibration process were the values obtained on the mechanical characterization performed on the same experimental test campaign, adjusted to match the desired results presented in Figure 1. Moreover the specimen considered as the most representative of a mixed shear-flexure behaviour was a 3.0 meters long wall (3.0 x 0.3 x 2.75 m^3) with a total vertical load of 300 kN which was used during the calibration process. After this calibration, the calibrated parameters were tested to reproduce the results obtained on the other tested walls and also good agreement was obtained. The final model parameter values are presented in Table 1, where E is the modulus of elasticity, G is the shear modulus, ρ_m is the density, f_m is the masonry compressive strength, f_{vo} is the shear strength under zero compressive stress, G_c is the non-linear deformability parameter, μ is the friction coefficient, β is the softening parameter, $\delta_{flexure}$ and δ_{shear} are the rocking and shear ultimate drift ratio. More information regarding these parameters may be obtained on the recommended bibliography.

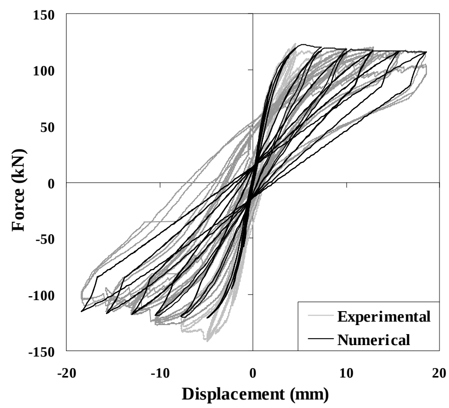
Table 1. Model parameters

E (MPa)	G (MPa)	ρ_m (kg/m^3)	f_m (MPa)	f_{v0} (MPa)	G_c	μ	β	$\delta_{flexure}$	δ_{shear}
1600	416	484	2.2	0.125	1.2	0.05	0.05	0.6	0.7

The final calibration curves obtained with the parameters presented in the previous table yielded to good agreement with the experimental test results (Figure 1), both for the cyclic response and the hysteretic energy dissipation.



(a)



(b)

Figure 1. Experimental vs. numerical results (N = 300 kN): (a) 1.5 m wall; (b) 3.0 m (Costa, 2007)

3. SEISMIC PERFORMANCE ASSESSMENT

3.1. Buildings characteristics

In addiction to the reproduction of the experimental results with a numerical model, the main objective of this work was to simulate and assess the seismic performance of different AAC masonry buildings with different structural typologies. Therefore 6 different buildings were analyzed aiming to represent some recommended prescriptions of the Italian Seismic Code (OPCM no. 3274, 2005) concerning the percentage of load bearing walls. The following Figure 2 represents the recommended ratios and the ratios of the assessed buildings.

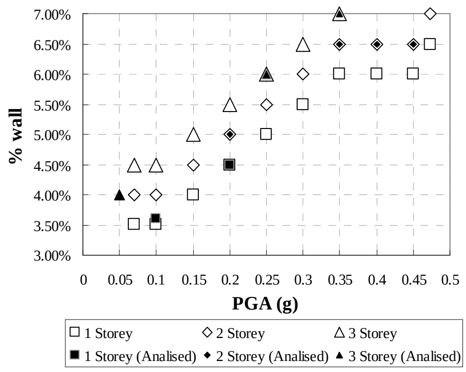


Figure 2. Recommended % of load bearing walls: prescribed in the Italian Seismic Code (OPCM no. 3274, 2005); analysed buildings

Aiming to clarify the typology of the buildings as also as structural configurations, Figure 3 presents the bare structure of each building as also as the direction of the performed analyses.

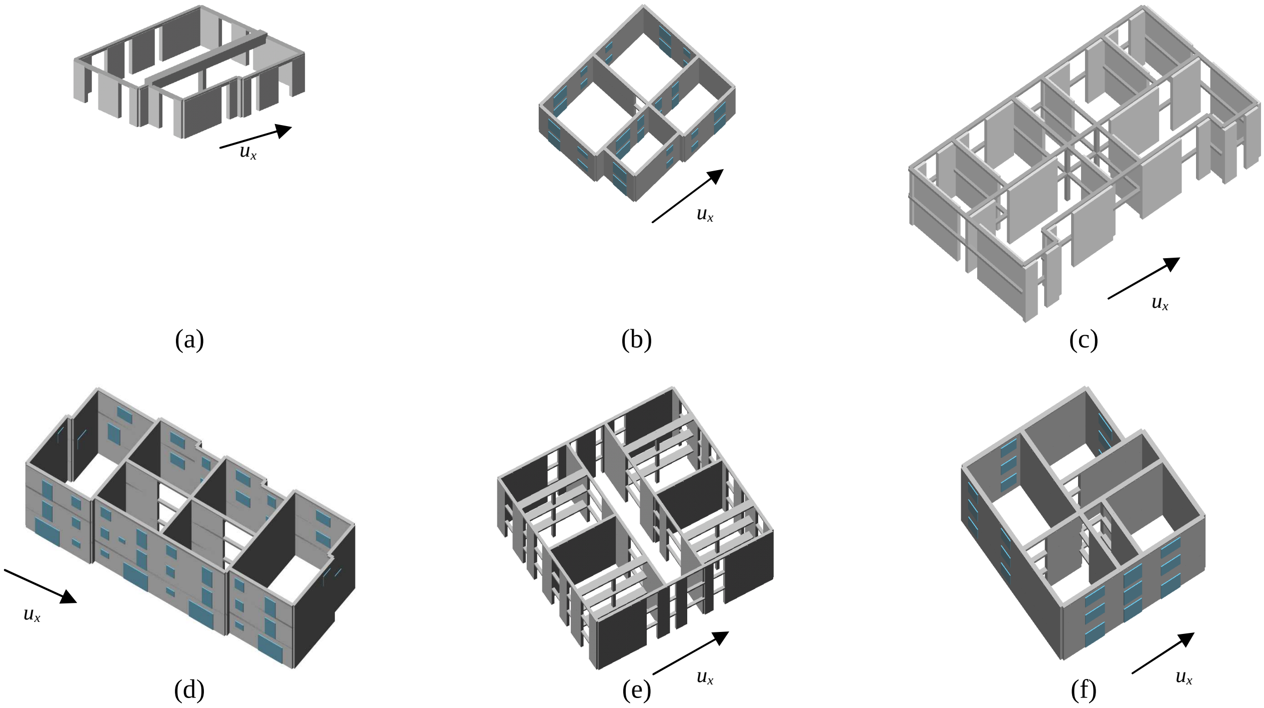


Figure 3. Bare structure of the analyzed buildings and direction of analysis: (a) Building 1 (wall (x) = 4.4 %); (b) Building 2 (wall (x) = 6.5 %); (c) Building 3 (wall (x) = 5.0 %); (d) Building 4 (wall (x) = 6.0 %); (e) Building 5 - (wall (x) = 4.0 %); (f) Building 6 (wall (x) = 7.0 %).

As it is possible observe in the previous Figure 3, different buildings' configurations were analyzed regarding the distribution in plan as also as the presence of reinforced concrete vertical elements (building 1, 3 and 5). It should be referred that the slabs are made of lightweight reinforced concrete with reinforced concrete tie beams at edges to correctly connect the horizontal element to vertical masonry panels. Another point that should be noticed is that all analysed structures represent real/designed Italian buildings enhancing the objective of this work.

3.2. *Type of performed analyses*

Aiming to correctly characterize the seismic performance of the presented AAC masonry buildings, two different approaches were used and compared to infer their applicability on seismic performance assessment.
So the nonlinear static procedure recommended in Eurocode 8 and Italian Seismic Code based on the capacity response method, consistent with Fajfar approach (Fajfar, 2000), was used and discussed with the other followed approach, the non linear dynamic analyses for different PGA levels. However the current building codes typically consider modal and uniform load distribution to perform the pushover analysis yielding to the theoretical capacity curve, which may not reproduce correctly their dynamic behaviour through a single analysis. Therefore the called Seismic Displacement-based Adaptive Pushover (SDAP) proposed by Galasco *et al.* (2006) and implemented in TREMURI program was used, where the actual deformed shape is evaluated at each step of the pushover analysis and it can be used to set the force ratios at the next step. This can be described as:

$$\begin{aligned} \{f_0\} &= p_0[\mathbf{M}]\{\boldsymbol{\psi}_\mathbf{0}\} \text{ starting step} \\ \{f_1\} &= p_1[\mathbf{M}]\{X_\mathbf{0}\} \text{ step 1} \\ \{f_2\} &= p_2[\mathbf{M}]\{X_1\} \text{ step 2} \\ \{f_i\} &= p_i[\mathbf{M}]\{X_{i-1}\} \text{ step i} \end{aligned} \tag{3.1}$$

where for the generic i-step $\{f_i\}$ is the force ratio, p_i is the participation coefficient, $[\mathbf{M}]$ is the mass matrix, $\{\boldsymbol{\psi}_0\}$ is the initial first mode eigenvector and $\{\mathbf{X}_i\}$ is the current deformed shape. The force ratios computed in the current step cannot exceed the two fixed boundary distributions (modal and uniform). Hence the force distribution is normalised and then compared with the normalised boundary distributions. If one or more components exceed the boundary values, then the component value is set to the normalised boundary one. This process is iteratively repeated for each component. The failure condition has been assumed to correspond to a 20% decay of the maximum force attained during the analysis, as recommended by Eurocode 8 and the Italian Seismic Code.

The nonlinear dynamic analyses were performed with TREMURI program (3D time history dynamic analysis implemented with Newmark integration method and Rayleigh viscous damping) based on the Italian Seismic Code proposal (similar to Eurocode 8) which refers that to correctly assess the seismic behavior of structures, 7 different accelerograms compatible in their mean values to the code's spectra should be used. On the presented work, each building was analysed for each peak ground acceleration (PGA) value compatible with the Italian hazard zones (0.05 g, 0.15 g, 0.25 g and 0.35 g) as also as for the maximum PGA level prescribed by the code (0.49 g = 1.4 x PGA, for fundamental structures). The input collection of real earthquakes was conveniently scaled to obtain an average acceleration response spectrum consistent with soil type A of Eurocode 8 (the same adopted in the Italian Seismic Code).

4. COMPARISON AND DISCUSSION OF RESULTS

In this section the obtained results will be presented in two different parts: presentation and comparison between pushover and dynamic results, aiming to assess the efficiency of the Seismic Displacement-based Adaptive Pushover (SDAP) algorithm to reproduce the dynamic behaviour of masonry structures; comparison regarding displacement demand vs. PGA level obtained through the capacity response method and dynamic analyses, with the objective of assess the seismic behaviour of AAC masonry buildings and evaluate the applicability of simplified procedures to estimate the dynamic response for these type of structures.

4.1. Pushover analyses

The results obtained with the pushover procedure reported in 3.2 are presented in Figure 4 in terms of mean top drift (δ) versus acceleration a ($a = F/m$), where F is the total base shear and m is the total mass of the building. Superimposed to these results are presented also the dynamic results obtained for each analysis and peak ground acceleration levels.

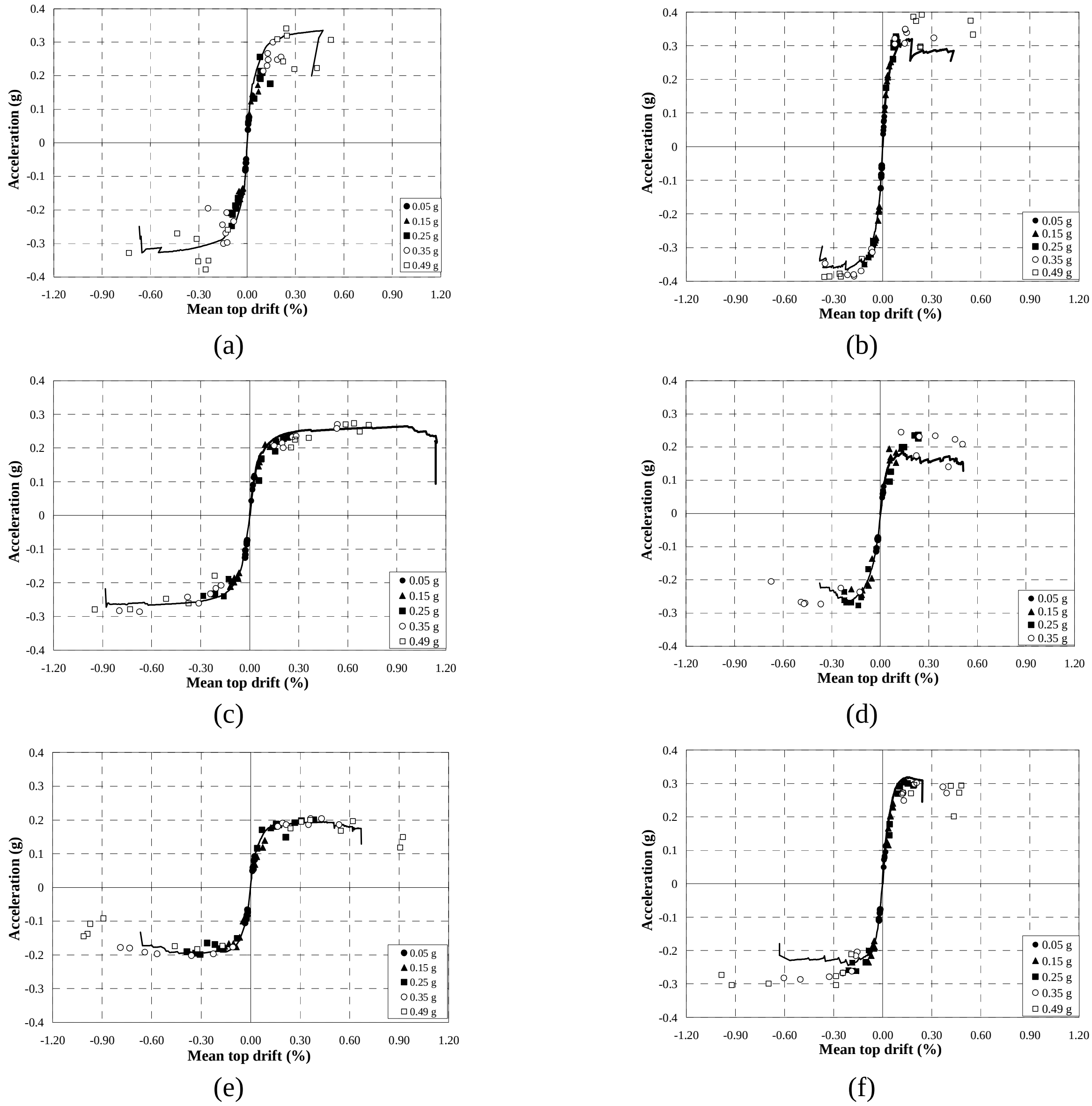


Figure 4. Pushover curves superimposed with dynamic analyses results: (a) building 1; (b) building 2; (c) building 3; (d) building 4; (e) building 5; (f) building 6.

From the figures above it is possible to conclude that the nonlinear static pushover curves and the points representing maximum force displacement bins resulting from nonlinear dynamic analyses are in good agreement in all the analysed structures. Therefore it is possible to infer that the adaptive pushover procedure

developed by Galasco *et al.* (2006) is able to envelope correctly the dynamic response and a correct estimation of the displacement demand using the capacity response method may be expected.
What regards the nonlinear behaviour of those buildings, in general they possess good nonlinear displacement capacity, especially those with regularity in plan and façades (building 3 and 5). On the other hand the non-equally distributed opening in plan and elevation may cause damage concentration leading to smaller maximum ultimate drift levels. Finally, the geometrical irregularity in plan may cause significant torsional effects reducing building's displacement capacity. The displacement ductility (μ_Δ) capacity obtained through a bilinear idealization of the response proposed by Costa (2007), as it will be explained in 4.2, lead to mean values higher than 5.

4.2. Nonlinear static predictions

The capacity response method prescribed by the Italian Seismic Code (similar to Eurocode 8 approach) aims to predict the displacement demand of the dynamic response of a structure through a simplified methodology able to reproduce the dynamic behaviour. Therefore, for each building, the direction of the pushover curve with lower displacement capacity was selected as representative of a global behaviour.
Then the pushover curve is transformed into an equivalent bilinear curve which aims to reproduce the response of the m.d.o.f structure by a s.d.o.f. elastoplastic oscillator based on an energy equivalence criteria. However in these analyses two different approaches to compute the bilinear idealization were used: Eurocode 8; Costa (2007). The former is based on the equal energy criteria up to the maximum strength of the structure; the latter takes into account also the equal energy criteria considering the total energy dissipated up to the collapse (20% decay of the maximum strength) and a stiffness degradation to define the equivalent yielding point. The main difference between approaches is the definition of the equivalent stiffness (K_e) which implies substantial differences on equivalent elastic period (T^*) and displacement ductility (μ_Δ) values. More information regarding the last procedure may be consulted in Costa (2007).
Table 2 presents a comparison between displacement ductility values obtained for each capacity curve of Figure 4 using the described approaches, where significant differences may be observed.

Table 2. Displacement ductility obtained for each capacity curve following Eurocode 8 and Costa's proposal

			Building					
			1	**2**	**3**	**4**	**5**	**6**
Displacement ductility (μ_Δ)	**+ *x***	Eurocode 8	3.8	6.2	6.6	7	7	6.5
		Costa (2007) proposal	8.5	6.1	10.9	8.5	9.4	10.9
	- *x*	Eurocode 8	4.5	4.5	4.7	3.7	7.1	4.7
		Costa (2007) proposal	14.4	5.8	6.5	3.7	7.1	6.5

Finally, the displacement demands were computed for each building and each Italian hazard zone PGA levels. Figure 5 presents the displacement demand estimated with the simplified nonlinear static method (with the two proposals to compute the bilinear idealization of the response presented before) and the nonlinear dynamic analyses, represented in terms of d/d_u versus peak ground acceleration (PGA), where d is the displacement demand and d_u is the minimum ultimate displacement observed from both senses of the capacity curve.

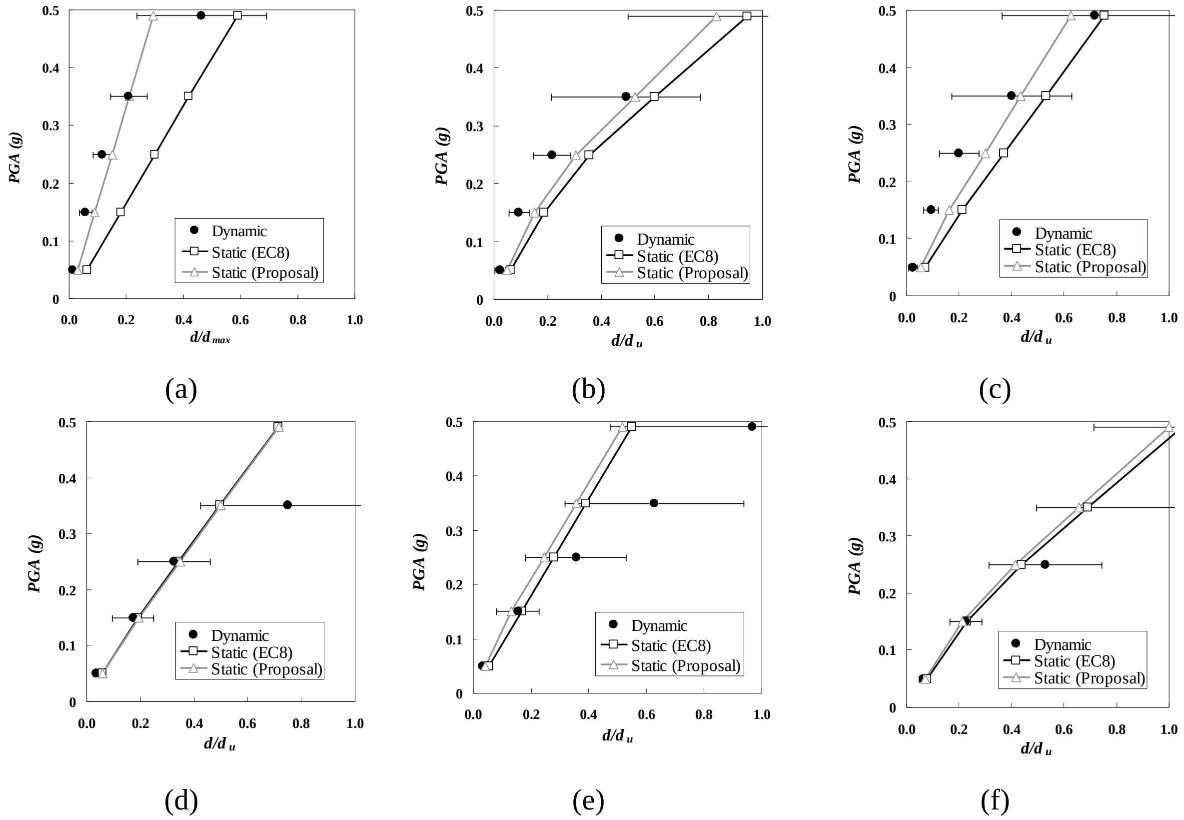


Figure 5. Displacement demand obtained with nonlinear static prediction and nonlinear dynamic analyses: (a) building 1; (b) building 2; (c) building 3; (d) building 4; (e) building 5; (f) building 6.

The figure above permits to infer that the Costa's proposal may be consistent and give more reliable results regarding displacement demand when compared with the Eurocode 8 approach. When the displacement demand is not coincident between the two approaches, the displacement demand computed with the bilinear proposed by Costa is more approximated to the dynamic response (e.g. Figure 5 a, b and c).
Regarding the efficiency of the capacity response method to predict the dynamic response of AAC masonry buildings, it should be referred that for small to medium PGA values (0.05 g ≤ PGA ≤ 0.25 g) it may give consistent results and may be used to predict the displacement demand of such buildings.

5. CONCLUSIONS

Despite the summarized description of the work performed, it is possible to infer some interesting conclusions regarding the seismic assessment of AAC masonry buildings as also as the numerical tools used in the presented work. The calibrated nonlinear macro-element model correctly reproduced the cyclic tests results on AAC masonry panels regarding hysteretic behaviour and envelope of the response which permitted to assess the seismic performance of complete three-dimensional AAC masonry building models. Therefore the nonlinear adaptive procedure (SDAP) provided capacity curves well matching the nonlinear dynamic envelope curves in all analysed buildings, enhancing the capability of Galasco *et al.* (2006) adaptive algorithm to reproduce the nonlinear dynamic behaviour through a nonlinear static procedure. In addiction to that, the suggested bilinear idealization of the response by Costa (2007) reproduced more accurately the displacement demand obtained with nonlinear dynamic analyses than the procedure drafted in Eurocode 8 (CEN, 2005). Indeed for small to medium peak ground acceleration levels (0.05 g ≤ PGA ≤ 0.25 g), the nonlinear static predictions using Costa (2007) proposal lead to adequate or conservative displacement demands in 4 of the 6 analysed buildings, where the other 2 remaining results (building 5 and 6) were slightly not conservative.
The results of AAC building seismic analyses point out that for 1-2 storey buildings and for low to medium levels of excitation (for rock sites), the damage limit state may not be attained. For higher levels of excitation, significant damage can be expected especially for multi-storey buildings. No direct correlation was found

between % wall and buildings' behaviour in the performed analyses but geometrical regularity in plan is suggested to improve AAC structures performance.

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