Seismic performance assessment and upgrade design of a generic Adobe building

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ABSTRACT
Buildings made of adobe (unbaked masonry) material are particularly vulnerable to seismic excitation. The adobe material has typically low compression strength and stiffness with respect to baked masonry blocks. In this paper, generic adobe buildings located in zones of moderate and high seismicity are studied. For comparison, under the same working assumptions, a low-quality Italian masonry building is considered. The in-plane behavior of the considered structures is modeled by applying the *equivalent frame* approach; whereas, the out of plane behavior is modeled based on the rigid body out-of-plane overturning mechanism. It is observed that for structures the critical mechanism is the overturning. As an upgrade strategy, the use of chains is proposed. In the case of the generic frame located in high seismicity zone, the structure cannot withstand also the required in-plane performance. This was addressed by increasing the connectivity through the use of reinforced mortar. It is also observed that the configuration of the openings in the walls can affect their in-plane resistance.

1 INTRODUCTION

In the present century alone, nearly three million people have been killed by strong earthquakes. The majority of these people lived in densely-populated poor areas (Blondet *et al.*, 2003). Buildings made of adobe material (un-fired, or unbaked, bricks) are very common in low-income areas due to their low cost and relative simplicity in construction; the popularity of adobe material is partly due to its excellent thermal and acoustic properties. However, the adobe buildings can be categorized as risk-prone even under their own weight. In particular the large mass and low strength of these buildings make them vulnerable to earthquake action (Tolles III and Krawinkler, 1990). This structural type is also affected by adverse climate change effects; such as, the erosion due to rain and the prolonged contact with the water in case of flood. These phenomena often lead to an overall deterioration of the structural resistance.

The objective of this paper is to illustrate that it is possible to assess and to mitigate the seismic performance of adobe buildings by using simple and low-cost instruments. This is done by proposing a relatively simple methodology for modeling and assessment of the seismic behavior of adobe structures.

In order to model the in-plane behavior of the adobe building, the equivalent frame model was adopted (Magenes et al, 2000; Paticier et al, 2007). To model the out-of-plane behavior, the rigid-body limit analysis for over-turning is employed.

The retrofit design is carried out based on simple concepts available in literature.

The proposed methodology is illustrated for two case-study generic one-storey buildings located hypothetically at sites having reference seismicity of Iran and Italy, namely, high and moderate-high seismicity.

While adobe buildings are quite common in rural areas in Iran, in Italy it is more likely that poor masonry buildings exist. Characteristics of this latter kind of structures are applied, for comparison, to the same structural model considered for the Iranian case.

For each case-study, the seismic demand and capacity are evaluated. In cases where the structural performance is not adequate for either in-plane or out-of-plane behavior, simple retrofit strategies are proposed.

2 CASE STUDIES

2.1 *Seismic scenarios*

The seismic hazards considered are that of Grottaminarda in Campania Region (southern Italy) and Bam in Iran (Kerman region). These two sites characterize two zone of medium-high and very high seismicity. In fact, Grottaminarda was struck by the Irpinia Earthquake in 1980 (M 6.9) and Bam was destroyed by the 2003 Bam Earthquake (M 6.6). This choice is based on both hazard and exposure aspects, namely: (a) medium to high seismic intensity hazard and (b) high density of vernacular and adobe houses.

The seismic demand, in both cases, is defined through the acceleration response spectrum for a probability of exceedance of 10% in 50 years, which is the reference design spectrum corresponding to the life-safety limit state according to the corresponding national building codes: D.M. 2008 (CS.LL.PP., 2008) for Italy and ISIRI 2800 for Iran.

For consistency of the two cases, the local site classification is assumed to be moderately condensed soil. This translates into: soil type C and soil type III according to Italian and Iranian codes, respectively. The site's topographic condition is assumed to be planar in both cases. The design spectra are plotted in Figure 1.

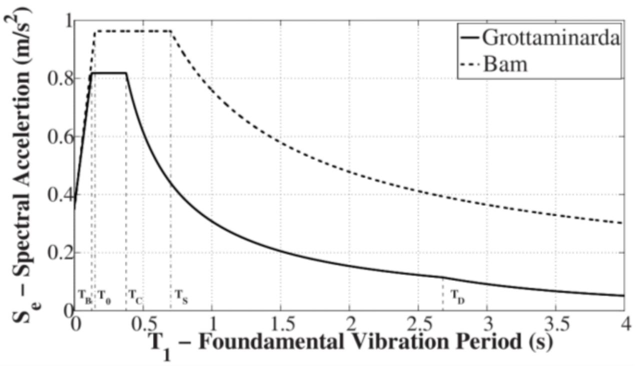

Figure 1 - Acceleration spectra for Grottaminarda (hard line) and for Bam (dashed line).

2.2 *Layout of prototype structures*

According to a recent survey study conducted by the Organization for Economic Co-operation and Development (OECD, 2009), about 77.4% of Italian territory is consisted of rural areas. In these areas, it is common to encounter one-story vernacular structures, usually built with low-quality masonry by owners for agriculture activities. In Iran, it is estimated that there are four million rural houses. Thirty percent of them is categorized as earthen buildings; eighty percent of which is estimated to have a flat roof, fifteen percent is estimated to have an inclined roof, and the remaining five percent has the arched roof. It is also estimated that eighty-five percent of the earthen buildings is one storey (Mousavi Eshkiki et al, 2006 – Ghannad et al, 2006).

The geometry of the case-study structure is established based on the above-mentioned statistics. The plan and elevation for the two alternative configurations of the case-study structure are illustrated below.

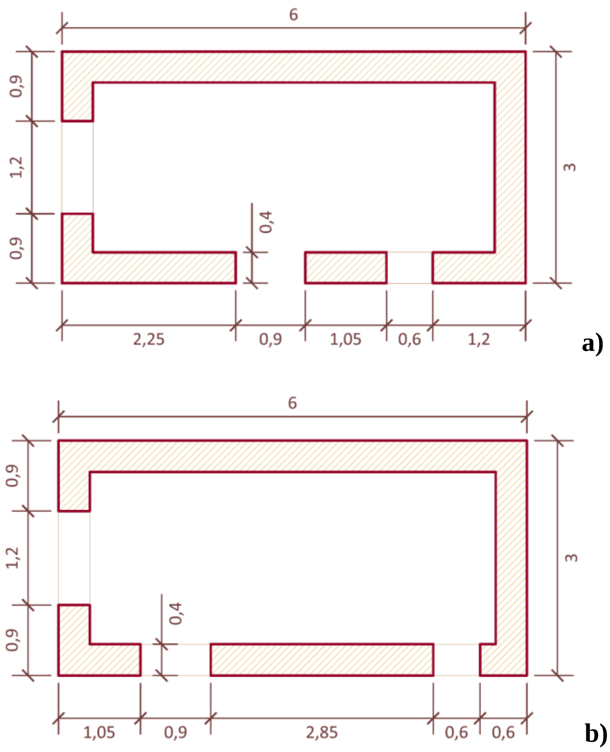

Figure 2 - Plan view: a) first configuration, b) second configuration, for the openings in the longitudinal wall. (unit: meters).

The difference between the first and second configuration lies basically in the alternative positioning of the openings in the longitudinal direction. In the first configuration, the openings are located in the central part of the wall;

whereas, in the second configuration, the position of the openings is less central with respect to the previous case.

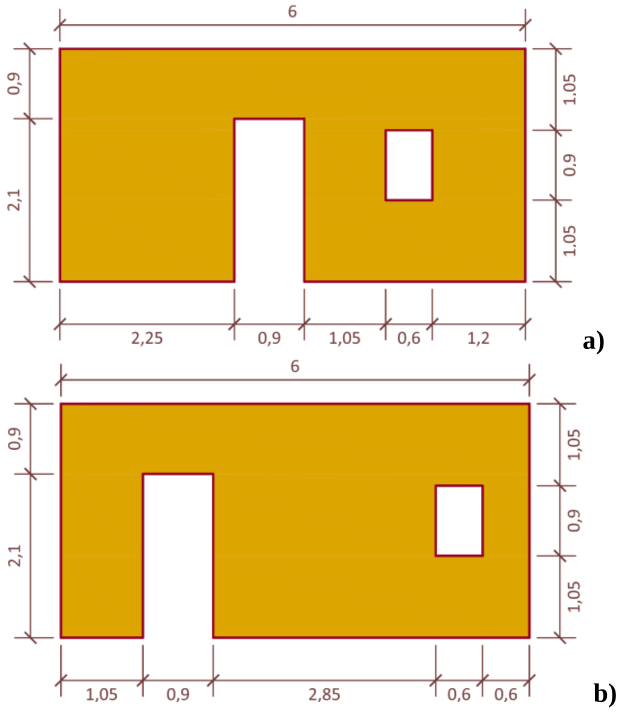

Figure 3 - frontal view: a) first configuration, b) second configuration, for the openings in the longitudinal walls. (unit: meters)

These two configurations are studied in order to verify the influence of the position of openings on the overall structural performance.

2.3 *Material properties*

In lieu of specific test results, the material properties for the Italian case-study are based on the code-based recommendations for low-quality masonry. The material properties for the Iranian case-study are obtained from test results reported in the literature for typical Iranian adobe material (Ghannad et al, 2006). Table 1 below reports the material mechanical properties for the two case study structures. The following mechanical properties are considered: the mean compression strength f_m; the mean shear strength τ_0; the mean Young modulus E; the mean shear modulus G; and the density γ.

Table 1. Material mechanical properties. I) Italian "Adobe", II) Iranian Adobe.

Type	f_m (MPa)	τ_0 (MPa)	E (MPa)	G (MPa)	γ (kN/m^3)
I	1.40	0.026	870	290	19
II	0.49	0.030	150	50	18

3 STRUCTURAL MODELING

Two alternative analysis methods are used for the evaluation of the in-plane and out-of-plane behavior of the case-study structures. The in-plane behavior is modeled by performing a two-dimensional non-linear static (pushover) analysis by the Open System for Earthquake Engineering Simulation (OpenSees) Software (McKenna et al., 2004). The out-of-plane behavior was modeled using the rigid-body kinematic over-turning limit analysis: where the wall is modeled as a rigid-body that can rotate around its base.

The in-plane model was constructed based on the equivalent frame concept (Magenes et al, 2000 – Paticier et al, 2007). This has made it possible to take into account in-plane failure mechanisms such as, combined compression and bending, shear with diagonal cracking and sliding shear.

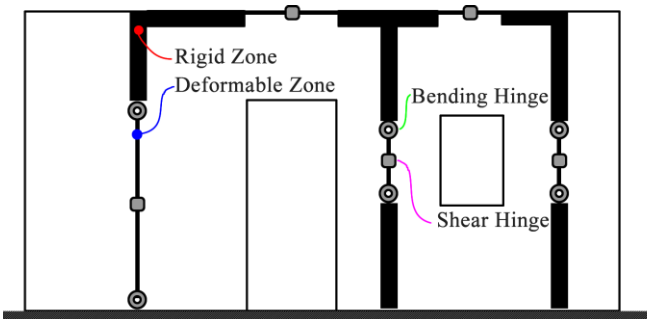

Figure 4 - The equivalent frame concept.

Figure 4 describes the equivalent frame concept in a schematic manner. The equivalent frame consists of mono-dimensional pier and the spandrel beam elements. The intersection of the piers and the spandrel beams is modeled as rigid. The non-linear behavior is modeled employing the lumped plasticity concept. It is considered that the structure is fixed at the base (as far as it regards the in-plane behavior).

The concentrated plasticity is modeled using the hysteresis material in OpenSees whose force-displacement curve is illustrated in Figure 5a. In particular, the flexural and shear behavior in the masonry piers are modeled as elastic-plastic with vertical strength drop (Figure 5c). This is accomplished by introducing two flexural hinges at the two ends of the deformable zone in the pier and one shear hinge at the mid-height of the deformable zone in the pier. The flexural and shear hinges are both modeled as rigid-perfectly plastic with vertical brittle strength-drop (Figure 5b). The spandrels beams are also modeled as elasto-plastic with vertical strength drop (Figure 5c).

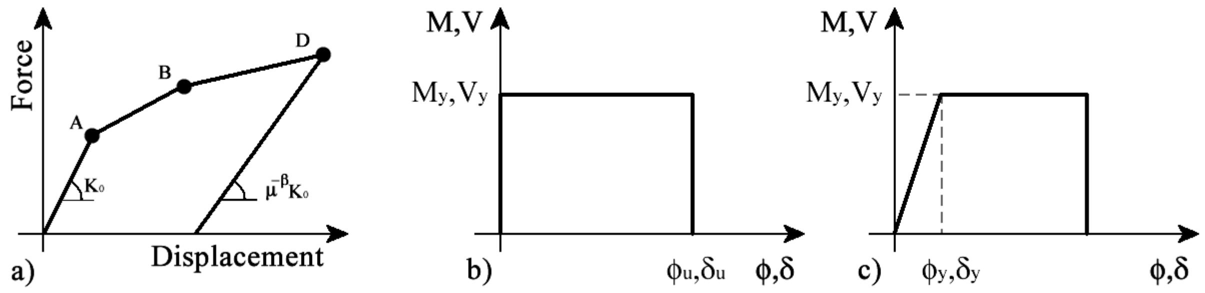

Figure 5 - a) standard force-displacement, b) force-displacement behaviour of the plastic hinge, c) force-displacement behaviour of the generic element.

The spandrel beams in this study can be characterized as deep beams based on their dimensions. Therefore, it is expected that they demonstrated a "shear type" brittle failure. As a result, only the shear plastic hinges are considered in their modeling (Figure 5b). It is worth mentioning that the flexural ductility of the spandrel beams are usually considered in cases where sufficiently anchored lintel beams are used.

The flexural strength of the pier denoted by (My) is defined trough the following equation:

$$M_y = \frac{N \cdot B}{2} \cdot \left(1 - \frac{N}{N_U}\right) \tag{1}$$

where N is the axial force (in compression) in the section, B is the wall length and N_U is axial strength of the wall in compression that can be evaluated from the following equation:

$$N_U = 0.85 \cdot t \cdot B \cdot f_m \tag{2}$$

where *t* is the wall thickness. As far as the shear strength is concerned, two alternative failure criteria are considered: (a) shear failure with diagonal cracking, originally proposed by Turnšek and Cacovic (1971), and (b) sliding shear failure.

The equations for the diagonal cracking shear and the sliding shear strength, respectively, are shown below:

$$V_y = t \cdot B \cdot \frac{1.5 \cdot \tau_0}{b} \cdot \sqrt{1 + \frac{N}{1.5 \cdot \tau_0 \cdot t \cdot B}} \tag{3}$$

$$V_y = \frac{1}{2} \cdot N \cdot \frac{3 \cdot \tau_0 \cdot t \cdot B + 0.8 \cdot N}{3 \cdot \alpha \cdot \tau_0 \cdot t \cdot H + N} \tag{4}$$

where H is the length of the deformable zone. α and b are calculated as follows:

$$\alpha = \frac{M_{MAX}}{M_{MAX} + M_{MIN}} \tag{5}$$

$$b = \begin{cases} 1.5 & \text{if} \quad H/B > 1.5 \\ H/B & \text{if} \quad 1.0 \le H/B \le 1.5 \\ 1.0 & \text{if} \quad H/B < 1.0 \end{cases} \tag{6}$$

where M_{MAX} and M_{MIN} are the bending moments at the two ends of the deformable zone in the equivalent frame element. The two types of shear failure, namely, the diagonal cracking and the sliding shear are modeled as two separate hinges in series. Assuming negligible axial forces, the shear strength for the spandrel beam is given by the following equation:

$$V_U = h \cdot t \cdot \tau_0 \tag{7}$$

where *h* is the spandrel depth.

As far as it regards the deformation limits for the pier elements, the ultimate plastic rotation ϕ_u in the flexural hinges is assumed to be equal to 0.6%. The shear hinges are assumed to have ultimate plastic displacement δ_u equal to 0.4% of the deformable height of the pier. Both deformation limits are based on the recommendations of the Italian building code.

The shear hinges in the spandrel beam are assumed to have an ultimate plastic displacement δ_u equal to 0.2% of the deformable length of the spandrel reflecting a more brittle behavior of those elements.

4 ASSESSMENT PROCEDURE

The seismic assessment of the case-study structures in this work is carried out neglecting the *box* effect. This means that it is assumed that the connectivity and the rigidity provided in the corners is not sufficient to guarantee that the walls in the orthogonal directions deform together as one frame. Moreover, it is also assumed that the roof system is not capable of providing a diaphragm action. These assumptions all underline the fact that the majority of adobe buildings are constructed without engineering supervision and without considering seismic provisions regarding the continuity of the structural system.

Neglecting the box action means that it is assumed that the walls in orthogonal directions are analyzed separately without considering the support provided by the corners. For each wall, the critical failure mechanism is identified by comparing the in-plane and out-of-plane failure mechanisms.

4.1 In-plane response

The in-plane analysis for each wall is performed by employing the incremental static nonlinear analysis (pushover analysis). The results are normally reported in the form of the static pushover curve which has the base-shear plotted versus the top displacement. The activation of each hinge, for the case-study structures considered herein, translates into a vertical drop in the base-shear on the pushover curve.

The structural collapse is identified by the complete loss of load bearing capacity. This is while the structural instability is marked by the onset of a disproportionate increase in the top displacement.

In order to calculate the seismic capacity of the structure by employing the code-based spectrum, the seismic response of the multiple degree of freedom (MDOF) structural system is mapped into that of an equivalent single degree of freedom (SDOF) system with appropriate mass, stiffness strength and ductility. In this work, the properties of the equivalent SDOF system are obtained following the approach suggested by Shibata and Sozen (1976). To this end, the dynamic response of the MDOF system is assumed to be decomposed into a time-invariant mode-shape vector ψ and time-variant control displacement d and base shear V. Using asterisk to identify the characteristic of the equivalent SDOF system, the relation between the MDOF properties and equivalent SDOF properties are:

$$d^* = \frac{d}{\Gamma} \tag{8}$$

$$V^* = \frac{V}{\Gamma} \tag{9}$$

where Γ is the modal participation factor calculated as follows:

$$\Gamma = \frac{\psi^T M \ \tau}{\psi^T M \ \psi} = \frac{m^*}{\psi^T M \ \psi} \tag{10}$$

M denotes the mass matrix and τ denotes the directional unit vector.

Herein, a linear shape vector as the ratio of vertical height z to total building height H is adopted:

$$\psi = \frac{z}{H} \tag{11}$$

The control displacement D and the base-shear V are estimated by employing the N2 method (Fajfar, 2000).

4.2 Out-of-plane response

The kinematic limit analysis is performed by assuming that the wall panel behaves as a rigid body and can rotate around the axis at the base of the wall on the external side. The kinematic method is based on the principle that on the verge of instability due to overturning, the equilibrium conditions and the principle of virtual work is still satisfied. In this framework, it is possible to estimate the acceleration at which the wall overturning takes place. Figure 6 shows the schematic diagram of the equilibrium of forces in a wall panel. The diagram shows the service loads (in black) and the seismic inertial forces (in gray). Usually, the service loads include the self-weight of the wall and the loads applied at the top of the wall. The forces of inertia generated by the earthquake act as destabilizing forces.

Denoting by α_o the ratio of the inertial forces on the verge of overturning to the stabilizing service loads and applying the principle of virtual work, the following equation can be derived:

$$\alpha_0 = \frac{T \cdot z + P \cdot y^P + P^w \cdot y^w}{P \ \cdot z \ + P^w \cdot z^w} \tag{12}$$

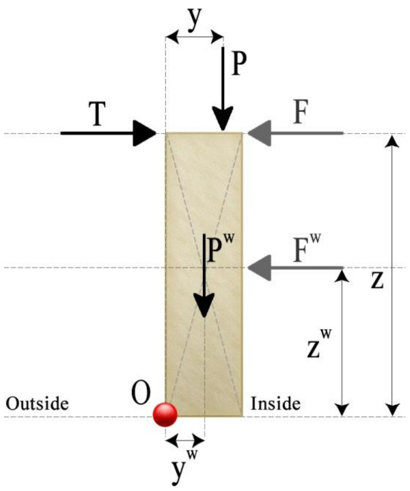

Figure 6 - Reference scheme for the kinematic analysis.

where P and P^w are the gravitational service loads and T is the transverse chains reaction (if present); z, y, z^w and y^w denote the arms of T, P, F^w and P^w, respectively. Φ denotes the virtual rotation of the wall panel as a rigid body. The participating mass to the overturning mechanism can be calculated as:

$$M^* = \frac{\left(P \cdot z + P^w \cdot z^w\right)^2}{g \cdot \left[P \cdot (z)^2 + P^w \cdot \left(z^w\right)^2\right]} \tag{13}$$

Having calculated the participating mass, the overturning acceleration, indicated as a_{OT}, can be calculated as:

$$a_{OT} = \frac{\alpha_0 \cdot \left(P + P^w\right)}{M^*} \tag{14}$$

The overturning mechanism is activated if a_{OT} is less than the peak ground acceleration.

4.3 *Comparison between out-of-plane and in-plane performance*

The results obtained for the in-plane and the out-of-plane behavior of the structure should be compared for both orthogonal directions. In order to be able to compare the in-plane and the out-of-plane performance for each direction, the out-of-plane overturning acceleration is marked on the pushover curve (plotted as acceleration versus displacement) for the equivalent SDOF system of the orthogonal wall.

If the overturning acceleration is less than peak ground acceleration, the out-of-plane overturning governs and the structure fails in a brittle manner. Otherwise, the structure would behave in a more ductile manner and the structural capacity will be equal to the in-plane capacity evaluated by the N2 method. In other words, for each direction, the out-of-plane performance of a given wall is compared with the in-plane performance of the orthogonal wall.

The overall structural capacity is evaluated by the minimum capacity value between the two orthogonal directions.

5 NUMERICAL RESULTS

In this section, the methodology described in the previous sections is employed for the performance assessment of two different configurations of the case-study adobe buildings (illustrated in Figures 2 and 3) located in the zones of moderate to high seismic hazard (Italy) and very high seismic hazard (Iran). In particular, the capacity curves expressed in terms of generalized displacement – acceleration (i.e., expressed in terms of the spectral displacement – spectral acceleration) are illustrated in Figure 7, Figure 8 and Figure 9. The results are classified for each direction, geometrical configuration and site location. The point called PP on the figure is the performance point obtained through N2 method. If the performance point exceeds the capacity point in terms of spectral displacement, the structure does not satisfy the required performance as far as it regards in-plane behavior.

On each curve, the overturning acceleration for the wall orthogonal to the one examined for in-plane behavior is plotted with a dash-dot line. It can be observed that the out-of-plane overturning is the critical mechanism for the structures examined. Moreover, the generic structural models located in Italy withstand seismic action as it regards the in plane (for both longitudinal and transversal direction) performance assessment for both configurations. This is while in Iran only the generic structure with the eccentric positioning of openings (the configuration "a" in Figure 2 and Figure 3) satisfies the in-plane performance requirements in the longitudinal direction. This can be attributed to the presence of a massive pier in this configuration that leads to more overall resistance for the wall.

In Tables 4, 5 and 6 R is the strength reduction factor (Miranda and Bertero 1994) and μ is the ductility.

Table 2. Results for the longitudinal direction for Grottaminarda Site (Italy).

Geo. Conf.	+ R	+ μ	- R	- μ	a_{OT} (g)
I	1.686	4.51	1.649	4.45	0.127
II	1.047	1.40	1.079	1.61	

Table 3. Results for the longitudinal direction for Bam Site (Iran).

Geo. Conf.	+ R	+ μ	- R	- μ	a_{OT} (g)
I	2.636	2.72	2.590	2.72	0.127
II	1.388	1.63	1.434	1.66	

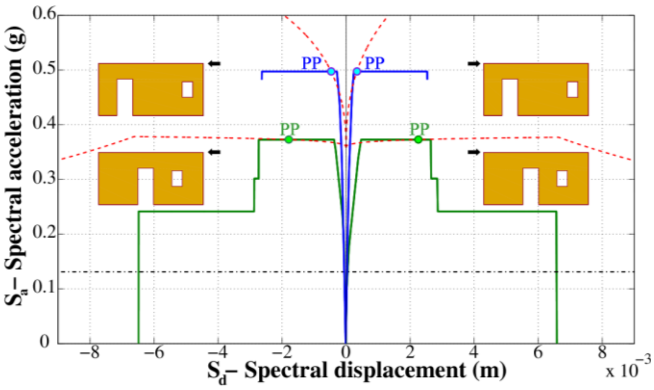

Figure 7 - Results relative to the longitudinal direction: Grottaminarda site.

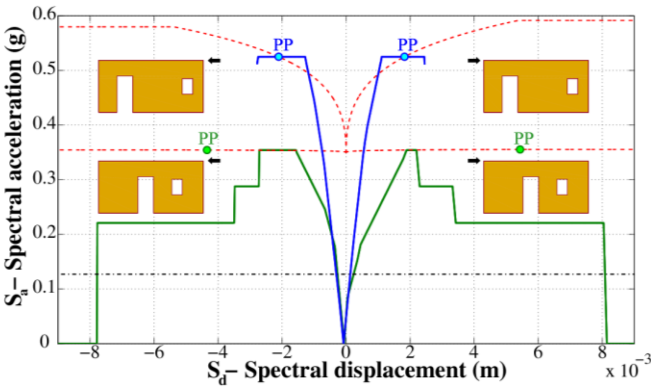

Figure 8 - Results for the longitudinal direction: Bam site.

Table 4. Results for the transversal direction both sites.

Grottaminarda R	Grottaminarda μ	Bam R	Bam μ	a_{OT} (g)
2.035	9.36	3.025	3.75	0.123

It emerges that the dominant in-plane rupture mechanism is the shear failure of the pier elements (i.e., a vertical drop in the wall resistance in the pushover curve).

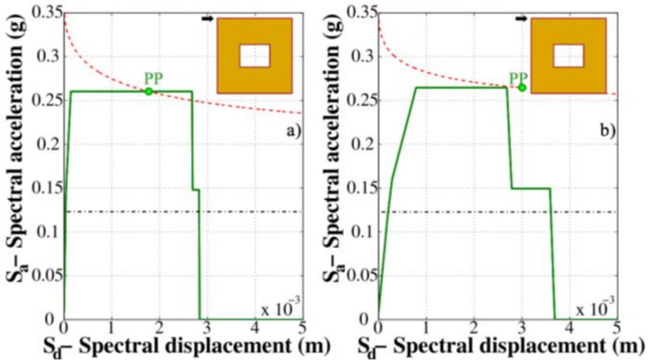

Figure 9 - Results for the transversal direction Grottaminarda (left) and Bam (right) sites.

6 RETROFIT SOLUTIONS

6.1 Chains

The retrofit solutions proposed in this section are calibrated based on the deficit of resistance illustrated in the two case-studies. Since the out-of-plane overturning emerges as the dominant failure mode, the retrofit solutions should address the out-of-plane resistance of the wall in the first place. In order to tackle this issue, it is possible to install chain elements between two opposite walls or to add a ring beam. The minimum strength of these elements can be evaluated by taking into account that the chains or the ring bream must compensate for the deficit out-of-plane resistance. In other words, as it can be observed from Figure 6, the resistance provided by the chain or the ring beam (denoted by T) would add up to the resistance due to self-weight of the wall in order to counter-balance the inertial forces. Therefore, the self weight multiplier a_0 for which the overturning acceleration α_{OT} is equal to the peak acceleration a_g can be calculated from Equation 14 and substituted back in Equation 12 in order to calculate the required axial resistance T:

$$T = \frac{a_g M^* \cdot \left(P \cdot z + P^w \cdot z^w\right) - \left(P \cdot y^P + P^w \cdot y^w\right)}{z \cdot \left(P + P^w\right)} \qquad (15)$$

The required axial force T can be used for designing the chain/ring beam element. In case, the simple chain is used, it is also necessary to verify both the punching resistance of the wall and the bearing resistance of the anchor plate as illustrated in Figure 10 below. σ_p is the interface pressure between the plate and the wall; σ_c is the punching stress; b and s are the width and the thickness of the plate, respectively.

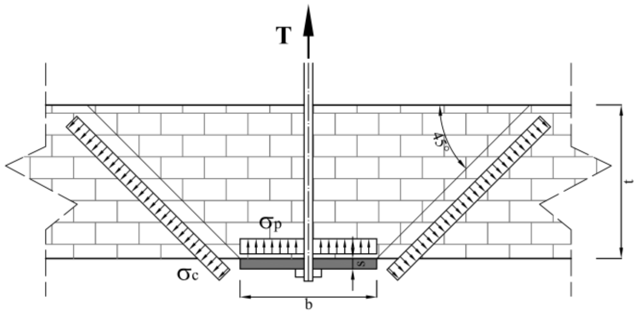

Figure 10 - Schematic diagram for the punching failure due to the chain element.

The required width (b) of the anchor plate necessary to prevent the punching shear failure can be calculated from Equation 16 below as the maximum width required taking in account the punching and crushing in the wall caused by the anchor plate. This is while the minimum required thickness s_{min} of the anchor plate can be calculated from Equation 17 as the thickness necessary prevent flexural failure in the critical section of anchor plate.

$$b = \max\left(\frac{T - 4 \cdot t^2 \cdot \tau_0}{4 \cdot t \cdot \tau_0}; \sqrt{\frac{T}{f_m}}\right) \tag{16}$$

$$s_{\min} = \sqrt{\frac{3 \cdot T}{4 \cdot f_{pd}}} \tag{17}$$

where f_{pd} is the flexural strength of the material with which the anchor plate is realized. Using a steel anchor plate with f_{pd} = 391.3 MPa and installing two chains in each wall, the required size of the chain and the anchor plate can be calculated from Equations 15, 16 and 17.

The resulting chain and anchor plate dimensions for longitudinal and transverse directions for both sites are reported in Table 7 below.

Table 5. Chain and anchor plate dimensions

SITE	Longitudinal direction			
	T (kN)	A_{min} (mm^2)	b_{min} (mm)	s_{min} (mm)
Grottaminarda	4.94	12.62	60	3.00
Bam	4.72	12.06	100	3.00
SITE	Transversal direction			
	T (kN)	A_{min} (mm^2)	b_{min} (mm)	s_{min} (mm)
Grottaminarda	2.36	6.03	45	2.15
Bam	2.25	5.75	70	2.00

6.2 Wire mesh

As mentioned above, the installation of the chains increases the resistance against out-of-plane overturning. Therefore, it is important to ensure that the upgraded structures can also warrant the required performance for the in-plane resistance. Recalling the results reported in the previous section, the generic structure located in Iran in its first configuration (Figure 2a and Figure 3a) needs to be upgraded for in-plane failure. In this work, it is proposed to adopt the reinforced mortar jacketing retrofit strategy. This solution, if employed on both sides of the wall and cross-connected by transversal bars, may lead to an increase of stiffness about 150%, as suggested by the Italian building code (CS.LL.PP., 2008).

Figures 11 and 12 below demonstrate the outcome of the retrofit application; the dashed line represent the pushover curves for the structure before applying the reinforced mortar and the solid lines represent the pushover curves for the structure after applying the reinforced mortar jacketing. It can be observed that the retrofitted structure is able to withstand the seismic demand for the in-plane performance assessment due to its increased stiffness.

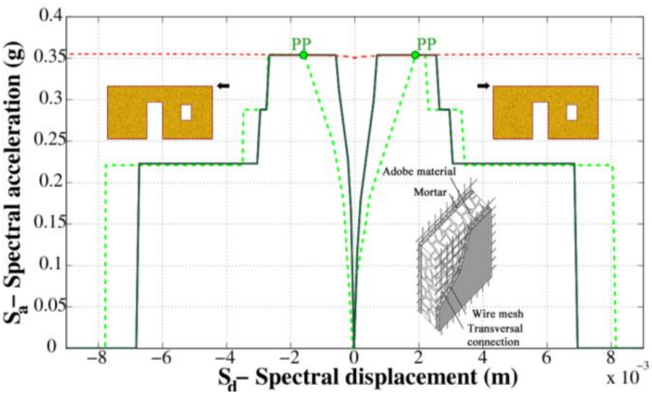

Figure 11 - In-plane performance assessment: longitudinal direction, Bam

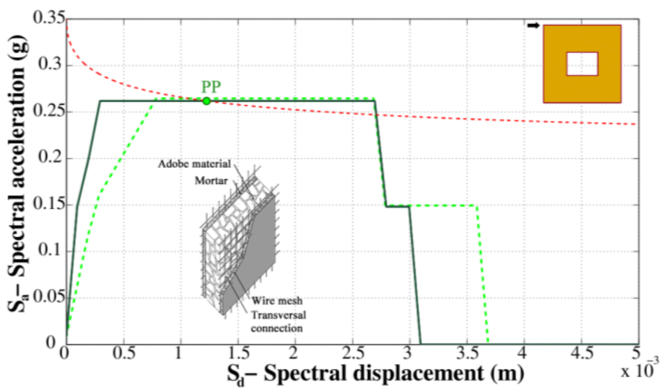

Figure 12 - In-plane performance assessment: transversal direction, Bam. The dashed lines represent the wall before

retrofit (reinforced mortar jacketing) and the solid lines represent the wall after retrofit

Tables 8 and 9 below summarize the N2 method results for both transverse and longitudinal directions of the structure located in Bam and corresponding to the first geometric configuration (Figure 2a and Figure 3a). It should be noted that the results reported in Table 8 for the longitudinal direction include both the position and negative loading consistent with Figure 11.

Table 6. Results for the longitudinal direction for Bam Site (Iran).

Geo. Conf.	+		-	
	R	μ	R	μ
I	2.636	2.01	1.934	2.710

Table 7. Results relative to the transversal direction for Bam Site.

R	μ
2.395	4.08

7 CONCLUSIONS

Adobe (un-baked brick) buildings are very common all over the world due to their low-cost and relatively simple construction techniques. However, these structures are particularly vulnerable to seismic excitations due to their low ductility and strength. In fact, from the examination of the adobe material mechanical properties, it emerges that the largest deficit is in terms of its compression strength and stiffness.

This paper aims to demonstrate that is possible to evaluate the seismic performance of this type of buildings using relatively simple analytical tools. Moreover, it is emphasized that simple and low-cost retrofit solutions can significantly improve the performance of the adobe buildings.

To assess the seismic vulnerability of this building typology, the generic adobe model is studied for two different sites: a moderate to high seismic hazard site located in Italy and a high seismic hazard site located in Iran. Two different geometric configurations are considered for the positioning of the openings in the longitudinal walls of the generic structure.

The proposed methodology employs the non-linear static analysis together with the N2 method in order to evaluate the in-plane performance of the individual walls modeled using the equivalent frame concept. The kinematic limit analysis is used for the evaluation of the out-of-plane behavior of the walls modeled as rigid body.

It is demonstrated that none of the generic structures in the two sites studied in this work can satisfy the required performance. In particular, it is observed that out-of-plane overturning is dominant for the case-study structures where a global box behavior is typically not present. It is also observed that, for the generic frame located in Iran, not even the in-plane performance requirements are satisfied.

Two simple and low-cost retrofit strategies, namely, the use of chains (increasing out-of-plane resistance) and reinforced mortar jacketing (increasing in-plane stiffness) are proposed. It is demonstrated that adopting these simple techniques leads to significant improvement in the over-all performance of the structure in terms of strength and stiffness.

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