Structural Behaviour and Retrofitting of Adobe Masonry Buildings

Humberto Varum, Nicola Tarque, Dora Silveira, Guido Camata, Bruno Lobo, Marcial Blondet, António Figueiredo, Muhammad Masood Rafi, Cristina Oliveira and Aníbal Costa

Abstract Earth is one of the most widely used building materials in the World. Different types of adobe dwellings are made to assure protection and wellbeing of the population according to the diverse zones needs. Therefore, it is important

---

H. Varum (✉) · D. Silveira · B. Lobo · A. Figueiredo · C. Oliveira · A. Costa
Civil Engineering Department, Aveiro University, 3810-193, Aveiro, Portugal
e-mail: hvarum@ua.pt

D. Silveira
e-mail: dora.silveira@ua.pt

B. Lobo
e-mail: bmlobo@ua.pt

A. Figueiredo
e-mail: ajfigueiredo@ua.pt

C. Oliveira
e-mail: cferreiraoliveira@ua.pt

A. Costa
e-mail: agc@ua.pt

N. Tarque · M. Blondet
Civil Engineering Section, Engineering Department, PUCP, 1801,
San Miguel, Lima, Peru
e-mail: sntarque@pucp.edu.pe

M. Blondet
e-mail: mblondet@pucp.pe

G. Camata
University "G. D'Annunzio", 65129 Pescara, Italy
e-mail: camata@unich.it

M. M. Rafi
Department of Earthquake Civil Engineering Department,
NED University of Engineering and Technology, Karachi 75270, Pakistan
e-mail: rafi-m@neduet.edu.pk

A. Costa et al. (eds.), *Structural Rehabilitation of Old Buildings*,
Building Pathology and Rehabilitation 2, DOI: 10.1007/978-3-642-39686-1_2,

to study the structural behaviour of the adobe masonry constructions, analysing their seismic vulnerability, which may help in preventing social, cultural and economic losses. In the present chapter, an explanation of the seismic behaviour of adobe buildings, a summary of recent research outputs from experimental tests conducted on adobe masonry components and from numerical modelling of full-scale representative adobe constructions are reported. In addition, different rehabilitation and strengthening solutions are presented and results from the testing of retrofitted adobe constructions and components are discussed.

Keywords Adobe • Masonry • Seismic vulnerability • Mechanical characterization • Numerical modelling • Retrofitting solutions

1 Introduction

Adobe derives from the Arabic word *atob*, which literally means sun-dried brick, being one of the oldest and most widely used natural building materials, especially in developing countries (Latin America, Middle East, north and south of Africa, etc.), many of which are also characterized by moderate to high seismic hazard. The use of sun-dried blocks dates back to approximately 8000 B.C., and until the end of the last century it was estimated that around 30 % of the World's population lived in earth-made constructions [1]. Adobe construction presents some attractive characteristics, such as low cost, local availability, the possibility to be self/owner-made with unskilled labour (hence the term "non-engineered constructions"), good thermal insulation and acoustic properties [2]. Adobe buildings present high seismic vulnerability due to the low tensile strength and fragile behaviour of the material, which constitute an undesirable combination of mechanical properties. Earthen structures are massive and thus attract large inertia forces during earthquakes; on the other hand, these structures are weak and cannot resist large forces. Additionally, this type of construction has a brittle behaviour and may collapse without warning [3]. The seismic capacity of an adobe house depends on the mechanical properties of the materials (blocks and joints), on the global structural system (structural geometry, connections, etc.), on building foundations, and also on the quality of the construction and maintenance [4]. Each time an earthquake occurs in a region with abundant earth-construction, enormous human, social and economic losses are recorded, as has been the case in El Salvador (2001), Iran (2003), Peru (1970, 1996, 2001 and 2007), Pakistan (2005), and China (2008 and 2009).

2 Adobe Constructions in the World

As previously said, 30 % of the World's population lives in earth-made constructions [1], with approximately 50 % of them located in developing countries. Adobe constructions are very common in some of the World's most

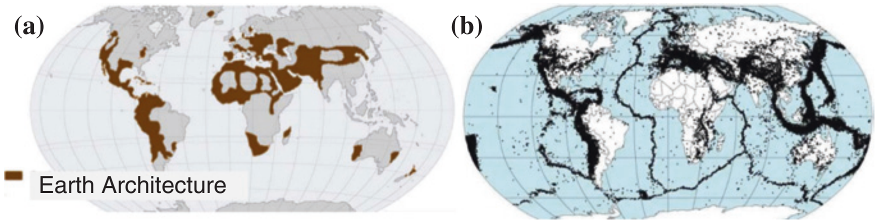


Fig. 1 Map of earthen constructions around the World and distribution of earthquake epicentres. **a** Distribution of earthen constructions [5]. **b** Earthquake epicentres (1963–1998) [6]

hazard-prone regions, such as Latin America, Africa, the Indian subcontinent and other parts of Asia, Middle East and Southern Europe [5]. Figure 1 compares the distribution of earthen constructions with the seismic hazard distribution in the World.

2.1 Typology of Adobe Dwellings in Southern Europe

A wide range of earth buildings can be found in Europe, particularly in Southern regions, as Portugal, Italy and Spain. But, in countries as Germany, England and France it can be also found a significant number of earth buildings. In France, for example, 15 % of the population lives in earth buildings, constructed with different techniques like adobe and rammed earth (‘*pisé’)* 0.

Italy, being one of the countries with the tradition in adobe construction, has a very pronounced historic and cultural earth built heritage. In old houses in urban areas, the load-bearing walls at ground floor are made mainly of stone masonry, in the intermediate floors walls are made of stone and/or adobe masonry, and in the upper floor walls are made of adobe masonry. In certain regions, the rural houses have their foundations made of stone and the walls of rammed earth or adobe masonry [7].

In Portugal, earth construction dates back to several hundreds of years ago, located mainly in the centre and south coastline [8–10]. These buildings are examples of the vernacular architecture, part of the Portuguese built heritage. In the district of Aveiro, a particular type of adobe was produced and used in construction. In this region, adobe blocks were stabilized with lime, to improve its mechanical strength and durability. In this region, the use of adobe in the construction was widely applied until mid of the twentieth century. After, the reinforced concrete as being used as the main building material/solution. The typical adobe houses in urban areas are two or three storey buildings (Fig. 2), sometimes covered with high valued tiling. In the city of Aveiro, it can be found several examples of buildings influenced Art-Nouveau architectural style [11, 12]. The majority of the constructions in rural areas are simpler and with only one storey, as can be seen in Fig. 3.

Fig. 2 Historical constructions in urban areas of Aveiro city

Fig. 3 Typical rural house in Aveiro surroundings

2.2 Other Typologies of Adobe Dwellings in the World

In South America, adobe is mainly used by low-income families. Adobe houses are predominant in rural areas, though they also exist in urban areas generally with a better quality of construction.

The typology of adobe dwellings is similar in most countries, with a rectangular plan, single door entry and small lateral windows. The walls are made with adobe blocks connected with mud mortar. The stucco is made with mud and sometimes mixing mud and gypsum. The foundation, if present, is made of medium to large stones joined with mud or coarse mortar. The roof supporting structure is made of wood joists resting directly on the walls or supported inside indentations on top of the walls. The type of roof covering depends principally on the family incomes and on the location of the constructions, and could be made of corrugated zinc sheets or clay tiles [13], this last preferable at the Peruvian highland.

In Peru, the percentage of the country's population living in earth dwellings has gone down from 54 to 43 % in the last 15 years [14] with most of the concentration in urban and coastline areas. Figure 4 shows some typical Peruvian adobe dwellings located at the countryside. But also in Lima, capital of Peru, some Colonial houses made of adobe can be found (Fig. 4c). These houses are typically characterized by a first floor made with adobe and clay bricks, and the second and third floors made with *quincha* (wooden frame with infill of mud and cane). Due to fragilities presented by adobe construction, some South American countries have been restricting its construction.

In Asia and Middle East (e.g.: Pakistan, Iran), see Fig. 5, adobe is a very traditional construction material. Luxurious adobe residences are constructed by wealthy families, and modest adobe houses by poor families.

In Iran, walls from rural constructions are usually built with adobe mixed with mud, stone, wood or bricks and concrete blocks [15]. In more than 4 million rural houses, at least 26 % have adobe and mud walls. Adobe dwellings in Iran are mainly characterized by the type of roof. The most common types are the vault

Fig. 4 Typical Peruvian adobe houses. **a** Adobe house located at the peruvian coast. **b** Adobe house in peruvian highland. **c** Colonial adobe houses of two and three storeys in Lima, Peru

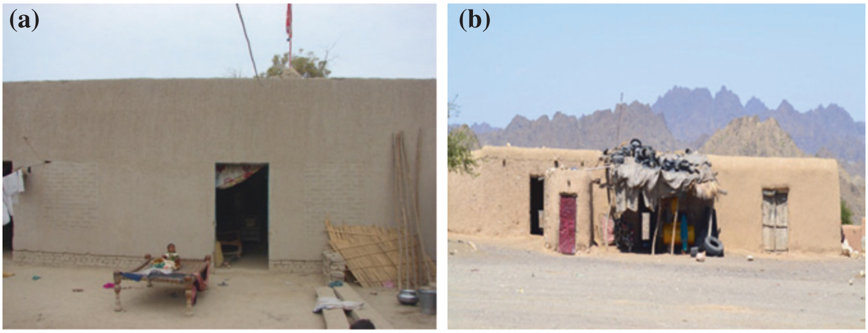


Fig. 5 Adobe houses in Pakistan. **a** Adobe house in Sindh. **b** Adobe house in Khuzdar (*Courtesy* Prof Sarosh Lodi)

and dome roofs. Vault roofs are built following a semi-cylindrical shape, with two plates or semi-spherical caps at their ends, being locally called '*tharby*'. When the vault covers only part of the roof it is called '*kalil*'. Other type of roofs are the quadripartite arched and crescent-arched roofs, in addition to the typical flat roof formed by wooden beams covered with branches of trees and mud.

In the northern regions of Africa, like in Morocco, the typologies of earth buildings vary from place to place. In Morocco, the Atlas chain divides the type of adobe dwellings. In the Drâa region, the constructions are built in fortified villages known as '*ksur'*. In these villages, the external walls are made from a technique called '*pisé'* (rammed earth), which allows the construction of very high walls, depending on the wall width. In this type of construction, adobe is used in the construction of columns and decorative elements both inside the patios and at the top of buildings [16].

3 Fragilities of Adobe Constructions

As stated before, adobe has been used in construction since ancient times due to cultural, climatic and economic reasons. Adobe structures can be in fact durable, but they present fragilities that have to be accounted for [9, 17].

Particular care should be taken with adobe constructions in high rainfall or humid areas, due to the susceptibility to of the materials to water and humidity. To avoid rising damp from the soil, an adequate solution at the foundation should be considered. Adobe masonry walls may also suffer erosion due to the wind actions. In order to protect adobe houses from the rain and winds, the covering of the exterior adobe masonry walls should be restored regularly to prevent the development of more severe damages, cracks and crumbling.

But, one of the most important fragility of adobe houses is related to their limited capacity to resist to earthquake demands, presenting poor behaviour for moderate to strong ground shakings, as observed in previous earthquakes, with important human losses and structural damages associated to adobe constructions.

As previously referred, the capacity of an adobe construction to resist earthquakes depends on the individual adobe block and mortar characteristics, on the mechanical behaviour and characteristics of the masonry system (considering blocks and joints), dimensions of the adobe wall (especially its thickness), building location and building geometry, as well as on the quality of construction and maintenance [4]. The extent of damage to an adobe structure depends on several factors such as: the severity of the ground motion; the geometry of the structure; the overall integrity of the adobe masonry; the existence and effectiveness of seismic retrofit measures; and the structure conservation state when the earthquake strikes.

The seismic vulnerability of adobe buildings is mainly associated to the perverse combination of the mechanical properties of their materials (low tensile strength and rupture in a brittle manner) with high density of their walls. As a consequence, every significant earthquake that has occurred in regions where earthen construction is common has produced life losses and considerable material damage (see examples in Fig. 6).

From the damage survey carried after the Peruvian earthquake of 2007 (Mw 7.9, 510 fatalities), it was concluded that the most common failure observed in non-reinforced earthen buildings, especially in those with slender walls, was the overturning of the façade walls and their collapse onto the street [18]. This happened because the effectiveness of the wall connection at the intersection between the façade wall and the perpendicular walls was too low to withstand the earthquake demands. The walls collapsed as follows: first vertical cracks occur at the wall's corners, originating damage in the adobe blocks in that area (Fig. 7). This triggered the walls to disconnect until finally the façade wall overturns. This is the most common collapse mechanism of adobe walls under earthquake actions. Observations made after the Peruvian earthquake have shown that the magnitude of damage suffered by the buildings was directly related to quality of the

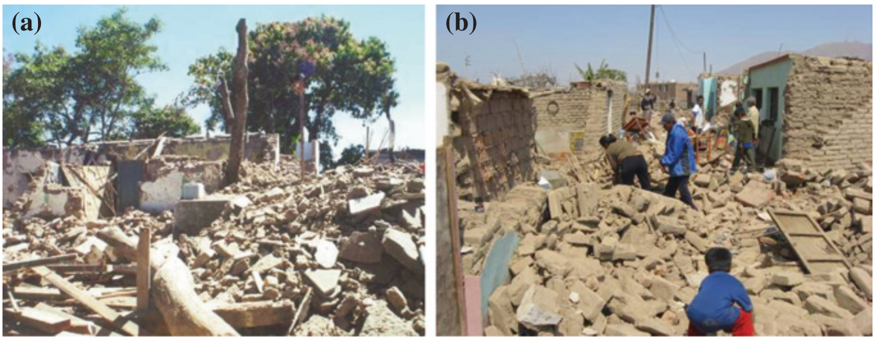


Fig. 6 Destruction of adobe houses due to earthquakes. **a** EI Salvad 2004 [4]. **b** Pisco, Peru 2007

Fig. 7 Vertical crack at the corner of an adobe house, Pisco earthquake, Peru, 2007

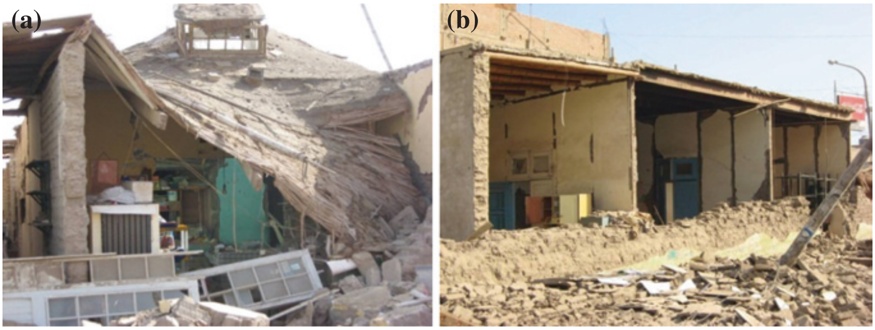


Fig. 8 Collapse of adobe houses during the Pisco earthquake in 2007, Peru. **a** Roof supported on the facade walls. **b** Roof supported by transversal walls

connection between the roof's wooden joists and the top of the façade walls. When the roof joists were supported by the façade wall, its collapse affects the roof support conditions, and finally the roof to collapse as well (Fig. 8a). If, on the other hand, the joists were supported by the walls that were perpendicular to the façade wall, the roof didn't fall apart (Fig. 8b).

Lateral seismic forces acting within the plane of the walls generate shear forces that produce diagonal cracks, which usually—but not always—follow stepped patterns along the mortar joints. The diagonal cracks often start at the corners of openings, such as doors and windows, due to the stress concentration at these locations (Fig. 9). If the seismic movement continues after the adobe walls have cracked, the wall breaks into separate pieces, which may collapse independently in an out-of-plane mode.

Dowling [4] makes a brief description of the common damage patterns of adobe dwellings based on a damage survey carried out after El Salvador earthquake (Mw 7.7, 825 fatalities) in 2001, where more than 200,000 adobe houses were severely damaged or collapsed.

Fig. 9 Typical X-cracks on adobe walls due to in-plane actions

As exemplified in Fig. 10, the more common damage patterns can be summarized as follows:

- Vertical cracking at corner angles associated to large relative displacement between orthogonal walls. This type of failure is very common because demands are largest at the wall–wall interface. Cracking occurs when the material strength is exceeded in either shear or tension;
- Vertical cracking and overturning of upper part of wall panel. Bending about the vertical axis causes a splitting-crushing cycle generating vertical cracks in the upper part of the wall;
- Overturning of wall panel due to vertical cracks at the wall intersections. Here the wall foundation interface behaves as a pin connection, which has little strength to overturning when an out-of-plane force is applied. This type of failure has been seen in long walls without other lateral restraints along the wall;
- Inclined cracking in walls due to large in-plane demands, which generates maximum tensile stresses in directions of about 45° relatively to the horizontal;
- Dislocation of corner. Initial failure is due to vertical corner cracking induced by shear or tearing stresses. The lack of connection at wall corners allows greater out-of-plane displacement of the wall panels, which generates a pounding impact with the orthogonal wall. The top of the wall is subjected to larger displacements, which tends to cause larger pounding, thus inducing greater stresses that may lead to failure;
- Horizontal cracking in upper section of wall panels, and displacement and deformation of the roof structure;
- Falling and slipping of roof tiles.

Webster and Tolles [19] conducted a damage survey on 20 historic and 9 older adobe buildings in California after the Northridge earthquake (Mw 6.7, 60 fatalities), in 1994. They concluded that ground shaking levels between 0.1 and 0.2 g PGA are necessary to initiate damage in well-maintained, but otherwise unreinforced, adobes. This study confirms that the most typical failure mechanism is due to out-of-plane flexural damage (Fig. 11). These cracks initiate as vertical cracks at the intersection of perpendicular walls, extending vertically or

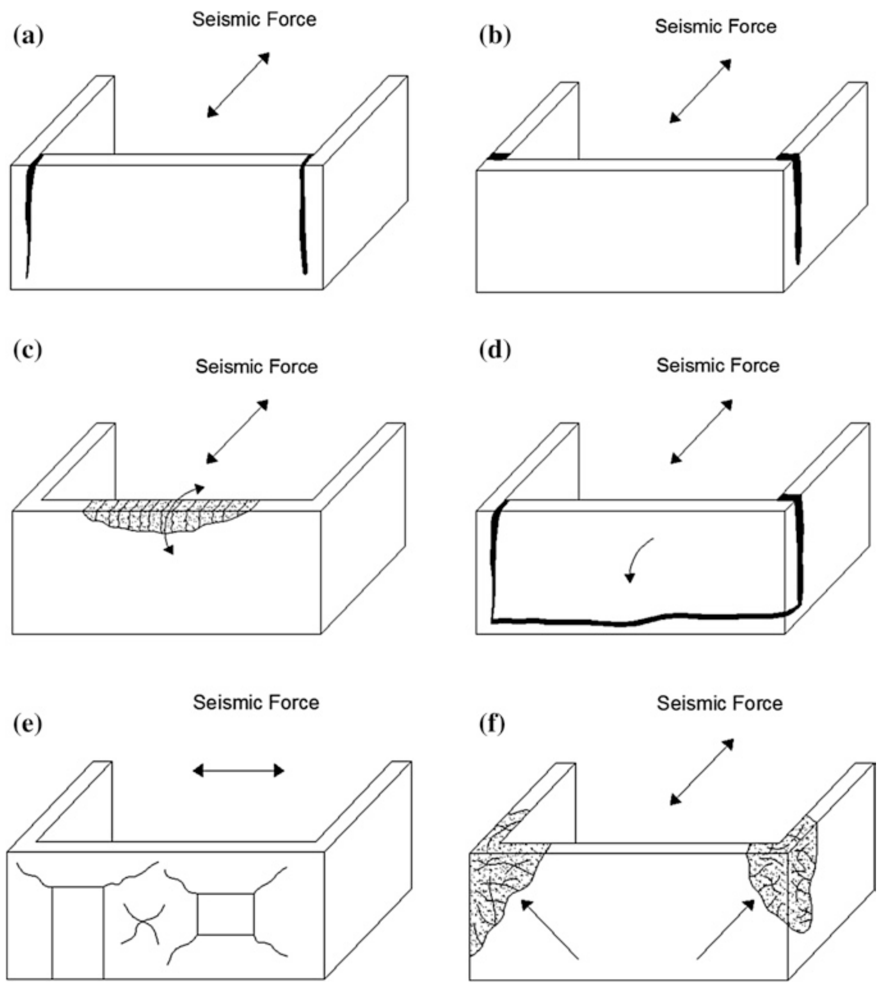


Fig. 10 Typical damage and failure mechanisms on adobe masonry constructions [4]. **a** Vertical corner angle cracking due to shear forces. **b** Vertical corner angle cracking due to to out-of-plane forces. **c** Diagonal cracking in wall due to in-plane shear forces. **d** Global overturning of wall panel. **e** Diagonal cracking in wall due to in-plane shear forces. **f** Sequence leading to corner dislocation

diagonally and running horizontally along the base between the transverse walls. Then, the wall rocks out-of-plane, back and forth, rotating round the horizontal cracks at the base. The gable-wall collapse is more specific for historic buildings. For long walls, the separation of these walls with the perpendicular ones results in out-of-plane moving of the wall. Diagonal cracks (X-shape) result from shear forces in the plane of the wall, and these cracks are not particularly serious unless the relative displacement across them becomes large. When the building is located at the corner of a building aggregate, some diagonal cracks appear at

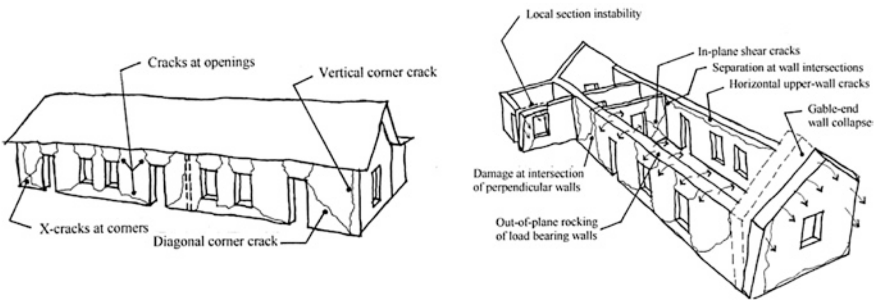


Fig. 11 Typical damages observed in adobe buildings after the Northridge earthquake [19]

exterior walls since they form wedges that can easily move sideways and downward as the building shakes, and also vertical cracks at the intersection of walls due to the out-of-plane actions.

4 Mechanical Characterization of Adobe Units and Masonry Panels

The properties of the adobe material are quite scattered and their values principally depend on the type of soil used for the fabrication of the blocks. In addition, the property values change if some binder or additive is included in the composition of bricks or mortar. Results of tests performed at two universities are described as follows showing the variation that can occur in terms of the material properties and mechanical behaviour of the adobe masonry.

4.1 Tests on Adobe Walls

4.1.1 Tests Performed at Aveiro University

A series of tests on 10 adobe masonry walls was developed at the Civil Engineering Department of Aveiro University. The adobe walls presented dimensions of $1.26 \times 1.26 \times 0.29$ m^3 and were built using adobe blocks from a demolition in Aveiro region and lime mortar formulated in the laboratory with a composition similar to the traditionally used [9].

Tests were performed according to the recommendations of ASTM E519 [20] and EN 1052-1 [21]. Five walls were tested in diagonal compression and the other 5 walls were tested in compression perpendicular to the horizontal joints. Tests were performed in a closed reaction frame using a servo-hydraulic actuator with a maximum load capacity of 300 kN to impose the displacements on the walls. The deformations of the walls were recorded during the test with a set of displacement potentiometers.

The adobe blocks used presented mean dimensions of 46 × 32 × 12 cm^3, specific weight of approximately 15 kN/m^3, mean compressive strength of 0.56 MPa and mean tensile strength of 0.13 MPa. A detailed analysis and discussion on the mechanical properties of the adobe blocks from the region of Aveiro can be found in [22–24].

Mean shear strength of 0.026 MPa and mean modulus of rigidity (shear modulus) of 40 MPa were obtained in the diagonal compression tests. The results in terms of stress versus deformation measured in both directions (vertical and horizontal) during the test are presented in Fig. 12.

The other five walls were tested in compression perpendicular to the horizontal joints. The scheme of the test and the distribution of instrumentation are presented in Fig. 13. From the tests, a mean compressive strength of 0.33 MPa and a mean modulus of elasticity of 664 MPa were obtained.

4.1.2 Tests Performed at the Catholic University of Peru

For the evaluation of the tensile strength, 10 square wallets of 0.6 × 0.6 × 0.2 m^3 were built using 0.2 × 0.40 × 0.08 m^3 adobe bricks, which imply 6 layers of 1 ½ adobe bricks [25]. The load was applied at two opposite corners of each wallet. Instrumentation to measure the diagonal deformations was applied in each adobe panel, and was used to compute the shear modulus. For a second group of tests [26], 7 panels were built and tested. More precise equipment for load application and to read deformations was used. From all the tests, the mean maximum shear strength stress was 0.026 MPa and the mean modulus of rigidity was 39.8 MPa, similar to the values reported by Aveiro University.

For the evaluation of the compressive strength of adobe prisms, a total of 120 samples were built by Blondet and Vargas [25] and Vargas and Ottazzi [26].

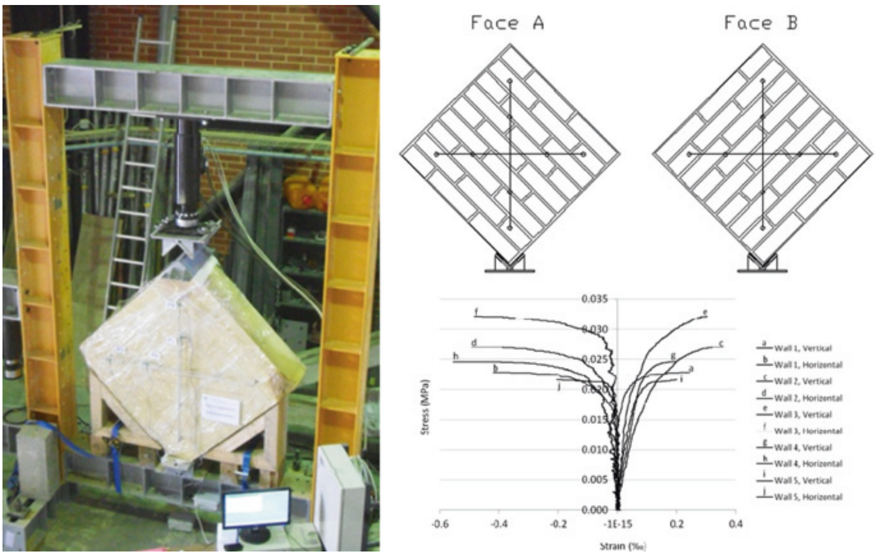


Fig. 12 Diagonal compression tests

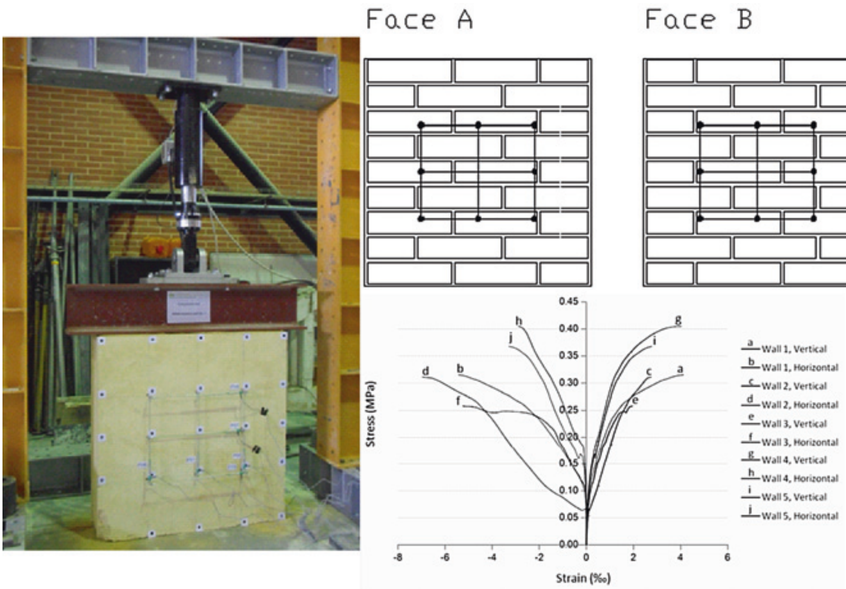


Fig. 13 Perpendicular compression tests

Specimens presented different slenderness ratios (thickness:height): 1:1, 1:1.5, 1:2, 1:3, 1:4 and 1:5. The adobe bricks had dimensions of 0.20 × 0.40 × 0.08 m^3, and were laid on top of each other with mortar in between: 89 specimens were built with mud mortar and 31 with a combination of cement, gypsum and mud mortar.

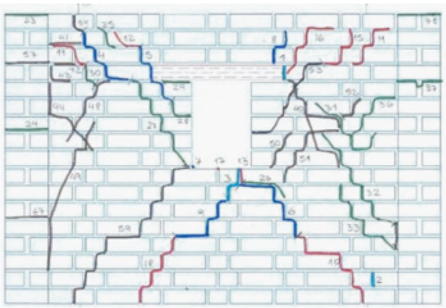
Only the tests on adobe prisms built with mud mortar are reported here. The irregularity of the top surface of each prism was corrected by adding a cement/sand mortar. Two steel plates of 0.20 × 0.40 × 0.02 m^3 were placed at both ends of each pile and were loaded axially. The axial load was applied perpendicularly to the joints with 2.45 kN increments until failure of the specimen. The test was force controlled. The axial deformation was measured in each prism tested. In all cases, the observed failure was brittle, and cracks did not follow a common pattern. As a preliminary conclusion it was established that the compression strength for prisms of slenderness 1:4 is between 0.80 and 1.20 MPa, depending on the specimen's age. The modulus of elasticity computed from full adobe wall tests was 170 MPa.

5 Tests on Full-scale Structures

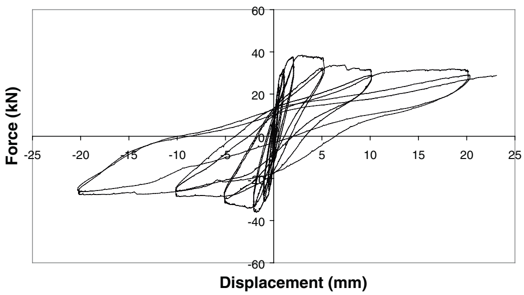
5.1 Double-T wall Tested at Catholic University of Peru

Blondet et al. [27] carried out a displacement controlled cyclic test (pseudo static push–pull) on a typical adobe wall at the Catholic University of Peru. With the first test it was intended to analyse the cyclic response of the wall and the damage pattern evolution caused by in-plane forces. The wall presented a double-T shape in-plan view (Fig. 14a), and the main longitudinal wall (with a central window opening) was 3.06 m long, 1.93 m high and 0.30 m thick. The structure also included two 2.48 m long transverse walls that were intended to: (a) simulate the influence of the connection between transversal walls found in typical buildings; (b) avoid rocking due to in-plane actions. The specimen was built on a reinforced concrete continuous foundation beam. A reinforced concrete beam was built at the top of the adobe wall to provide gravity loads corresponding to a roof composed of wooden beams, canes, straw, mud and corrugated zinc sheet. The top beam also ensures a more uniform distribution of the horizontal forces applied to the wall. The window lintel was made of wood. The horizontal load was applied in a series of increasing load cycles. Loads and displacements were applied slowly in order to avoid dynamic effects. Each displacement cycle was repeated twice. During the test, the cracks started at the windows' corners and advanced diagonally up to the top and down to the base of the wall. During reversal loads, the cracks generated the typical X-shape crack due to in-plane forces. Figure 14b shows the crack patterns that settled in the adobe wall during the tests. Considering, as control displacement, the lateral displacement at the base of the top beam, the cracks observed after two consecutive cycles for the displacement levels of 1, 2, 5 and 10 mm are marked in

(a)



(b)



(c)

Fig. 14 Cyclic test carried out on a double-T wall [27]. **a** Tested wall. **b** Damage pattern evolution during the cyclic test. **c** Cyclic test carried out on a double-T wall [27]

Fig. 14b. After an imposed displacement of 10 mm at the top of the wall, it was observed sliding of the adobe wall panels, as can be interpreted from Fig. 14c.

5.2 Dynamic Tests on Adobe Modules

A dynamic test analyses the response and the damage pattern evolution of adobe masonry structures when subjected to seismic actions. The unidirectional dynamic test was performed on an adobe module built on a reinforced concrete ring beam to simplify the anchorage of the specimen to the unidirectional shaking-table [3]. With this module it was intended to represent a typical Peruvian adobe construction located at the coast. The total weight (module + foundation) was approximately 135 kN. The weight of the concrete beam was 30 kN. The adobe bricks and the mud mortar used for the construction of the module had a soil/coarse sand/straw volume proportion of 5/1/1 and 3/1/1, respectively. The module consisted of four walls 3.21 m long, identified as right, left, front and rear wall. The thickness of the walls was 0.25 m, except for the right wall which had a thickness of 0.28 m because it was plastered with mud stucco (Fig. 15).

The adobe module was subjected to three levels of unidirectional displacement signals, which were scaled to present maximum displacements ate the base

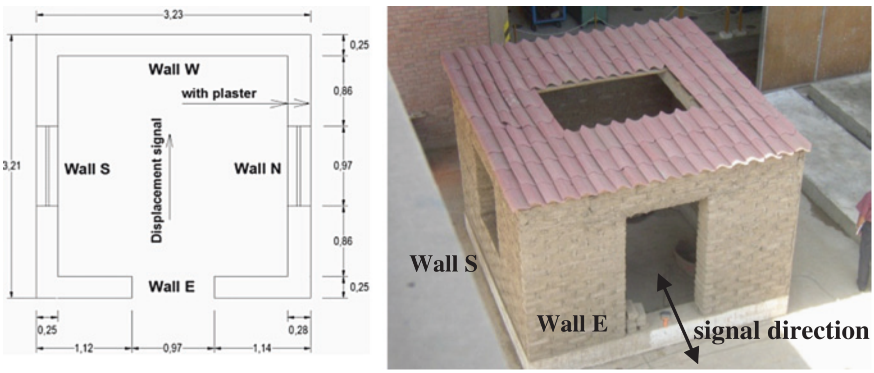


Fig. 15 Adobe module tested on PUCP shaking-table. **a** Plan dimentions. **b** 3D view

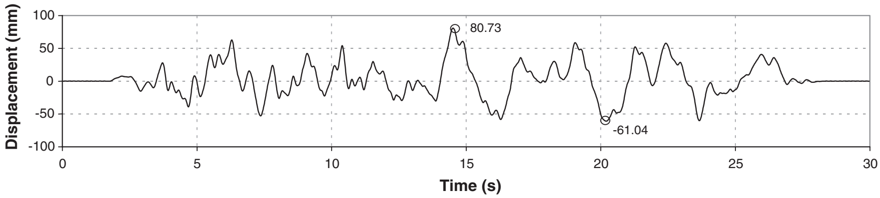


Fig. 16 Displacement input used in Phase 2 and scaled to present maximum amplitude of 80 mm

of 30 (Phase 1), 80 (Phase 2, Fig. 16) and 120 mm (Phase 3), to comparatively represent the effects of a frequent, moderate and severe earthquake. The input signals were scaled from an acceleration record of the Peruvian earthquake occurred in 1970.

At the end of Phase 1 and during Phase 2 typical vertical cracks appeared at the walls intersections causing the separation of the walls (Fig. 17), as typically occurs during moderate ground motions. Subsequently, X-shape cracks appeared at the longitudinal walls, and cracks appeared at the transverse walls due to horizontal and vertical bending. The anchorage of the steel nails that connected the wooden beams to the walls was lost during the movement, and as a result the roof was supported by the walls just through its own weight and friction. Major damage was observed at the end of Phase 2 and total collapse was observed during Phase 3.

(a)
(b)
(c)

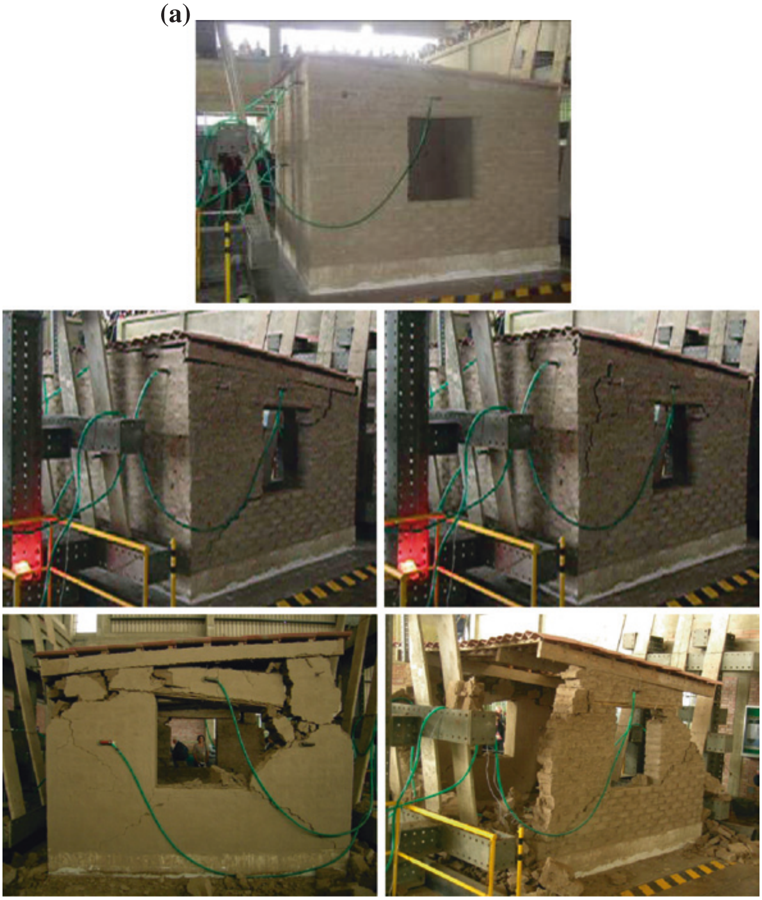


Fig. 17 Views of the adobe module during and after the dynamic test [3]. **a** Adobe odue after Phase 1. **b** Snapshots of the adobe module during Phase 2. **c** Adobe modue after Phase 3

Wall W, which was itself broken more or less into 3 big blocks (typical of walls supported only by three sides), presented a rocking behaviour due to out-of-plane actions. During Phase 3, with a maximum displacement at the base of 130 mm, the walls perpendicular to the movement (walls W and E) fell down at the beginning of the input signal, while the parallel walls S and N were completely cracked (Fig. 17). As the roof was supported by the lateral walls, it did not collapse. The formation of vertical cracks caused the separation of walls, allowing them to move independently.

5.3 Double-T wall Tested at Aveiro University

With the objective of conducting a thorough evaluation of the performance of adobe structures with and without seismic retrofit, an experimental study with a double-T wall was carried out at Aveiro University. A full-scale adobe wall was built in the Civil Engineering Laboratory using adobe blocks from a demolition in Aveiro region. A series of pseudo-static cyclic in-plane tests was carried out on the wall in order to evaluate and characterize the existing adobe construction in the region.

With the intention of considering the influence of adjacent walls, the wall was built in the shape of double-T in-plan view (Fig. 18a). The real-scale adobe wall presented the following dimensions: height of 3.07 m, length of 3.5 m and mean thickness of 0.32 m. The adobe blocks used in the construction of the wall presented mean dimensions of $24 \times 44 \times 12$ cm^3, specific weight (approximately) 15 kN/m^3, mean compressive strength of 0.42 MPa, and mean tensile of 0.14 MPa.

A vertical uniform load was added at the top of the wall through an equivalent mass of 20 kN to simulate the common dead and live loads on typical adobe constructions. A cyclic horizontal demand of increasing amplitude was applied 2.5 m above the base of the wall, until failure.

(a) **(b)** **(c)**

Fig. 18 Cyclic test on the double-T wall. **a** 3D View of the idealized adobe wall. **b** Final damage pattern of the original wall. **c** Final damage pattern of the strengthened wall

From the cyclic tests, the maximum lateral force obtained was 58.14 kN, with a corresponding shear strength capacity of 57.28 kPa and a maximum drift of 0.61 %.

After the first cycle, the wall's strength registered a strong decrease, with an important stiffness reduction. The failure mode was fragile, as expected for adobe constructions. An important factor for the decrease in the strength of a masonry wall is the strength capacity of the bond between the mortar and the adobe blocks. The initial development of cracks is mainly in the diagonal direction (Fig. 18b) [28, 29].

After the first experimental test series, the wall was repaired by pressure injection of a hydraulic lime gum into the cracks. Afterwards, the original plaster was removed, and a synthetic mesh was applied on the surface of the wall. The mesh was fixed to the wall with PVC angle pins and angle profiles, using highly resistant nylon thread on all concave vertices of the wall [28].

After repairing and retrofitting, the strengthened wall was tested using the same test procedure described for the original (non-strengthened) wall.

In the second test, the maximum shear strength of the wall was approximately 70.69 kPa with a corresponding force of 71.75 kN. The shear strength obtained for 1 % drift was approximately 45 kPa (70 % of its maximum shear strength) and the maximum imposed deformation was 1.6 % with a corresponding displacement of 45 mm. During the first cycles the wall response was almost linear, even though some small cracking occurred (Fig. 18b).

In order to evaluate the efficiency of the retrofit solution, a stress-drift plot was built with the results obtained in the two tests conducted on the original and retrofitted wall. With this plot it is possible to compare the wall responses before and after strengthening.

After repair and strengthening, the stiffness of the wall improved becoming very close to the stiffness of the original wall. The maximum resistant shear capacity of the wall increased 23.43 % after retrofit and the maximum deformation tripled.

The fragility of the wall post peak force decreased, and the ductility and energy deformation capacity increased. In consecutive cycles, a lower degradation of strength was observed.

The efficiency of the repairing and strengthening measures conducted on the wall was also evaluated by the observation of the values of the natural frequency of the wall before testing, in the original state of the wall and after reinforcement (Fig. 19b) [30].

By analysing the values obtained for the first frequency of the wall before and after retrofitting, it is clear that the rehabilitation process restored the original stiffness corresponding to the undamaged wall. The first frequency displayed in the graph corresponds to the wall before the beginning of the cyclic tests. The response of the retrofitted wall presented a smoother decrease of stiffness and consequently of natural frequency. Hence, it is possible to conclude that the repairing and strengthening measures performed are beneficial to the behaviour of the wall when subjected to horizontal displacements.

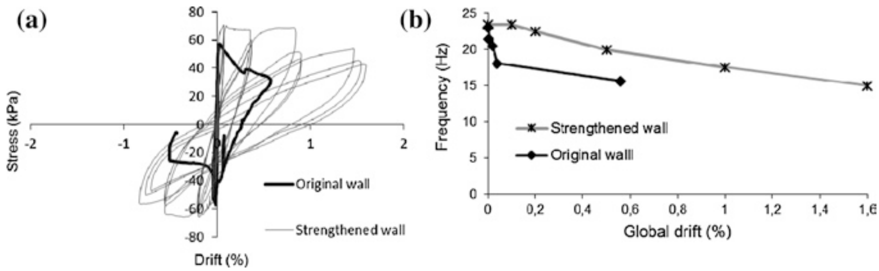


Fig. 19 Comparative results obtained in the test performed on the original and strengthened wall. **a** Comparative response of the original and strengthened adobe wall (stress versus drift). **b** First natural frequency evolution of the original and strengthened adobe wall

These significant improvements suggest that this solution can be used efficiently by construction and rehabilitation companies in the preservation and strengthening of existing adobe structures.

6 Numerical Modelling: Actual Knowledge and Needs for Research

6.1 Introduction

Masonry is a composite material made of bricks and mortar joints, each of the constituents with its own material properties. The level of accuracy in the numerical models strongly depends on the knowledge of the material properties (e.g. constitutive laws, isotropic or orthotropic behaviour, etc.), on the type of analyses conducted (e.g. linear, nonlinear), on the model used (e.g. shell elements, brick elements), and on the solution scheme adopted (e.g. implicit or explicit). Since adobe material is almost brittle, elastic analysis can give only information on the first cracking zones and not on the cracking process and cracking developing. In order to properly describe the seismic behaviour it is necessary to conduct nonlinear analyses.

The description of the nonlinear behaviour of adobe masonry is more complex than in the case of other materials (e.g. reinforced concrete, steel). The nonhomogeneous nature and variability of the material, the lack of information on the constituent material properties and the numerical convergence problems due to brittle behaviour are challenges that need to be overcome to properly analyse this material.

In addition, the cracking pattern observed in adobe walls subjected to horizontal loading is quite complex and difficult to predict. Generally, the mortar is weaker and softer than the bricks and therefore cracking tends to follow the mortar joints (Fig. 20a). However, sometimes failure of masonry may involve crushing and tensile fracturing of masonry units [31], in particular in adobe walls where the mortar has the same material properties as the bricks.

Fig. 20 Typical crack patterns in adobe masonry walls. **a** Stair crack shape (concentrated on mortar). **b** Vertical crack (fracture of bricks)

Detailed modelling of the masonry components, describing constituents separately, or modelling masonry as an equivalent and homogenous material are two possible options. Simplification by considering a homogeneous isotropic material for adobe is acceptable since adobe bricks and mortar are made of the same material, raw earth, and with the same binder when it is used.

6.2 Numerical Methods for Nonlinear Analysis of Adobe Structures

Numerical analysis of unreinforced masonry structures (URM) can be performed using different methods, such as: limit analysis, finite element method, discrete element method, amongst others [32–35]. Simplified approaches consist of idealizing the structure through an equivalent frame where each wall is discretized by a set of masonry panels where the nonlinear response is concentrated at the pier and spandrels [36–40]). In all cases the nonlinear information of the adobe material is important to describe properly the material behaviour. Each of the mentioned methods has advantages and disadvantages, and the analyst should adapt any of these methods according to his experience and expertise, computational facility available and data information. In the following sections some relevant methods are described.

6.3 Finite Elements Method

Following the finite element method, the analysis of masonry structures (e.g. clay brick, adobe, stone, etc.) can be classified according to the level of accuracy [33], Fig. 21:

- Detailed micro-modelling: Bricks and mortar joints are represented by continuum elements, where the unit-mortar interface is represented by discontinuous elements [41–44]. Any analysis with this level of refinement is computationally

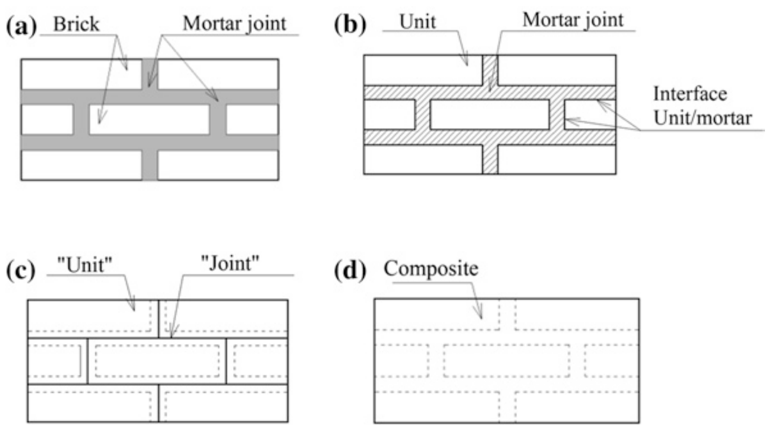


Fig. 21 Modelling strategies for masonry structures within a finite element approach [33]. **a** Masonry sample. **b** Detailed micro-modelling. **c** Simplified micro-modelling. **d** Macro-modelling

intensive and requires a well-documented representation of the properties (elastic and inelastic) of the constituents;

- Simplified micro-modelling: Bricks are represented by continuum elements, where the behaviour of the mortar joints and unit-mortar interface is lumped in discontinuous elements [34, 45–47]. This approach can be compared with the discrete element method, originally proposed by Cundall [48] in the area of rock mechanics, where a special procedure is used for contact detection and contact force evaluation [46];
- Macro-modelling or continuum mechanics finite element: Bricks, mortar and unit-mortar interface are smeared out in the continuum and masonry is treated as a homogeneous isotropic/orthotropic material. This methodology is relatively less time consuming than the previous ones, but still complex because of the brittle material behaviour.

The first two approaches are computationally intensive for the analysis of large masonry structures, but they accurately describe the behaviour of adobe and are an important research tool in comparison with the costly and often time-consuming laboratory testing. The third approach is faster than the previous ones and, in the case of adobe structures, does not significantly reduce the accuracy of the results. In the case of macro modelling, the selection of the nonlinear model used to represent the soil behaviour is very important to achieve accurate results (e.g. Mohr–Coulomb model, Drucker-Prager model, Concrete Damage Plasticity model, Smeared Cracking model, amongst others).

6.4 Discrete Elements Method

In the discrete element method the masonry structure is represented by an assembly of blocks with special nonlinear behaviour at their boundaries (e.g. mortar joints). The walls are modelled in a micro-scale level. This methodology

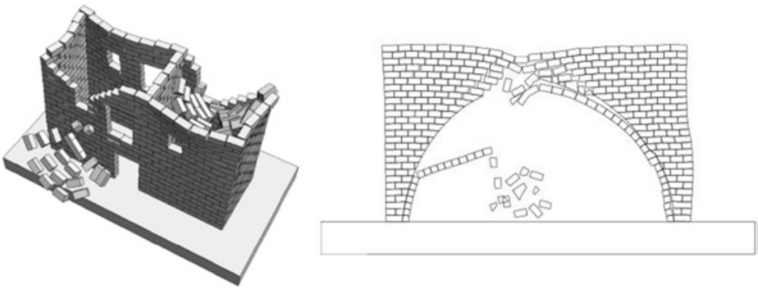


Fig. 22 Modelling of an unreinforced masonry structure (*left*) and a vaulted wall (*right*) within a discrete element approach [49]

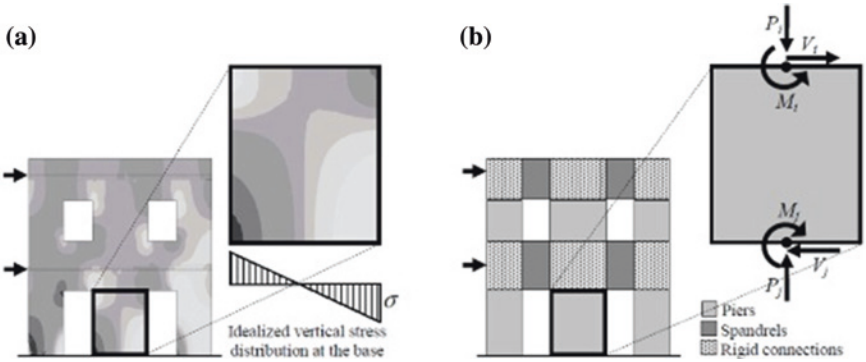


Fig. 23 Modelling strategies for masonry structures based on macro-elements [36]. **a** Modelling with FEM. **b** Modelling with the equivalent approach

allows the representation of large movements and complete block separation with results in changes in the structural geometry and connectivity as seen in Fig. 22. The algorithm recognizes new contact zones between the bodies (blocks, units, particles) as the analysis progresses [35]. As it is usual in a micro-scale model, the computational effort is demanding for the analysis of real adobe masonry structures. Normally, an explicit solution procedure is selected for the non-linear analyses.

6.5 *Approaches Based on Macro-models*

In the equivalent approach the masonry walls are modelled in a macro-scale level. This is a simplified method which can be used to evaluate the global strength of the building. Each masonry wall is represented by pier elements, spandrel beam elements and joint rigid elements (one-dimensional beam-column elements), as seen in Fig. 23. The piers are the principal vertical and horizontal seismic resistant elements; while the spandrels couple the piers in case of seismic loading.

The effective height of the pier should be able to represent well the non-linear behaviour of the panel. Nonlinear force-drift relationships represent the damage in the masonry panels due to flexion, shear and sliding. This equivalent approach is more accurate when the masonry walls are well connected by horizontal floors (not necessarily rigid floors) and the openings have a regular vertical distribution [40]. It is assumed that the global building response is more influenced by the in-plane response than the out-of-plane response of the walls. The out-of-plane is evaluated locally or it can be indirectly measured by the drift at roof level.

Each of the previous methodologies uses algorithm to compute the equilibrium at each step of the analysis. Those are called solution schemes and are divided into implicit and explicit procedures as it is explained in the next section.

6.6 Solution Schemes for Nonlinear Material Behaviour: Implicit and Explicit

One of the important issues in solving the nonlinear response of structures is the type of solution schemes used to indirectly evaluate the equilibrium of the system and the accuracy of the results. The solution of the problem is found by checking the convergence or the non-divergence of the structures under the application of incremental loading (force or displacement). There are two schemes: implicit and explicit.

6.6.1 Implicit

An implicit analysis is an iterative procedure used for checking the equilibrium in terms of internal and external forces of the system at each time step. This analysis implies the solution of a group of non-linear equations from time t to time $t + dt$ based on information of $t + dt$. For example, if $Y(t)$ is the current system state and $Y(t + dt)$ is the state at the later time, so the Eq. (1) should be solved to find $Y(t + dt)$.

$$G\,(Y\,(t),Y\,(t + dt)) = 0 \tag{1}$$

Amongst the different solution procedures used in the implicit finite element solvers, the Newton–Raphson solution is the faster intended for solving non-linear problems under force control. The convergence is measured by ensuring that the difference between external and internal forces, displacement increment and displacement correction are sufficiently small.

6.6.2 Explicit

Explicit solution was originally created to analyse high-speed dynamic events and models with fast material degradation (such as almost-brittle materials), which may cause convergence problems when analysed with implicit procedures. The explicit method solves the state of a finite element model at time $t + dt$ exclusively based on information at time t (Eq. 2). It implies no iterative procedure and no evaluation of the tangent stiffness matrix, which results in advantages comparing with the implicit procedure.

$$Y(t|dt) = F(Y(t)) \tag{2}$$

Here, the movement equations are integrated using the central difference integration rule, which is conditionally stable (this means the necessity of a small time increment). The stability limit for this integration rule is normally smaller than the Newton–Raphson procedure. The explicit stability limit is the time that an elastic wave spends to cross the smallest shell element dimension in the model.

6.6.3 Particular Aspects for the Modelling of Adobe Structures

In the case of adopting implicit solutions when analysing complex problems of brittle materials, such as adobe masonry, the algorithms to evaluate the nonlinear dynamic response may face convergence problems. In such cases, an explicit solution procedure could be used. The explicit modelling strategies can provide a robust method for large nonlinear problems. However, the accuracy of the model must be controlled, for example, by the crack pattern evolution and by the quantification of the energy (e.g. internal energy, external energy, etc.).

Normally, a non-linear analysis within an explicit scheme needs more running time that an analysis within the implicit one. This is because the time step for the explicit method should be small to avoid divergence in the results.

In addition, when dealing with quasi-static problems, like in pushover analyses, with an explicit scheme, the inertial forces should be reduced as much as possible. This procedure is not straight forward and some assumptions, like mass scaling, should be applied to the model. In this case (pseudo-static analyses), an implicit scheme could be more convenient for obtaining preliminary results of the numerical models [50].

7 Seismic Vulnerability Assessment of Adobe Constructions

7.1 Earthquake Loss Estimation

Earthquake loss estimation is considered one of the important components of disaster management programmes. In Chap. 1 this topic is addressed in more detail and in this section are just briefly introduced the general concepts and the results of recent research on earthquake loss estimation of adobe constructions. Loss estimation models can be used by the experts in the insurance industry, emergency planning and seismic code drafting committees [51]. The studies aimed at determining losses are based on earthquake risk, which is a product of hazard and vulnerability. Hazard refers to the probability of earthquake occurrence at a place within a specified time period, whereas seismic vulnerability refers to the potential damage of elements at risk. The elements at risk include buildings, infrastructures, people, services, processes, organisations, etc. [52]. For building infrastructure, vulnerability is expressed as expected damages to structures during ground shaking. The loss estimation studies can be carried out using either scenario studies or probabilistic analysis.

7.2 Seismic Hazard Analysis

Hazard is described in terms of a ground motion parameter such as peak ground displacement (PGD), peak ground acceleration (PGA), spectral displacement (Sd) and earthquake macroseismic intensity. The selection of a suitable parameter is dependent on the type of vulnerability analysis [53]. There are two approaches for carrying out a seismic hazard analysis: deterministic seismic hazard analysis (DSHA) and probabilistic seismic hazard analysis (PSHA). The size and location of the earthquake is assumed to remain unchanged in DSHA and the hazard evaluation is based on a particular seismic scenario. On the other hand, PSHA allows considering uncertainties in the size, location and rate of occurrence of earthquakes and variation in the ground motion due to these factors. Nevertheless, identification and characterisation of potential earthquake sources, which are capable of producing significant ground motion at a site, are common elements in both approaches. The characterisation of a source includes its location, geometry and earthquake potential. The determination of earthquake size on macroseismic intensity scale is based on human observations which are made during an earthquake regarding the damage of natural and built environment [54]. Different macroseismic scales are employed, such as, Modified Mercalli Intensity (MMI), European Macroseismic Scale (EMS-98) [55], parameter scale of seismic intensity (PSI scale) [56], etc. These provide a qualitative assessment of the effects of earthquakes on the building taxonomy, at a particular location.

7.3 Seismic Vulnerability Analysis

Vulnerability assessment methods provide a relationship between the intensity of ground shaking (hazard) and expected building damages in terms of mean damage grade (μ_D). This relation is termed as vulnerability curve. The results obtained from vulnerability curves can be extended to develop building fragility curves, which provide an estimate of conditional probability of exceeding a damage state of a building, or portfolio of buildings, under a given level of earthquake loading. In probabilistic terms, fragility is a cumulative density function (CDF) which represents the vulnerability of a building or building stock to failure [57].

Vulnerability analysis can be carried out with the help of building inventory for the area of interest. Different vulnerability assessment methods have been proposed in the published technical literature. These can be divided into four categories: empirical methods, judgement-based methods, mechanical methods and hybrid techniques. Of these, the first two are qualitative whereas the last two are quantitative methods. A brief description of these is given in the following paragraphs:

- Empirical Methods: These methods are based on the data of observed damage which is collected from the post-earthquake field surveys. These are the oldest seismic vulnerability assessment methods which were developed as a function of macroseismic intensities [58]. Different empirical methods include damage probability matrix (DPM) [59], vulnerability index [60], vulnerability curves [61], continuous vulnerability functions [62], etc.
- Judgement-based Methods: These methods employ the information provided by the earthquake engineering experts based on their judgement. The experts provide estimates of probability of damage likely to be experienced by different structure types at several ground shaking intensities. A judgement-based method was first employed by the Applied Technology Council (ATC) for California and a summary of the method is available in ATC-13 [63].
- Mechanical Methods: Nonlinear numerical analyses of computer models of the buildings are carried out for different intensity earthquakes. The information on damage distribution obtained from the analyses is statistically analysed to develop vulnerability curves. These methods are employed when the available earthquake damage data is insufficient.
- Hybrid Techniques: Different researchers, e.g. [64–66], have employed hybrid methods of seismic vulnerability assessment. The development of these is a result of deficiencies in the aforementioned methods in carrying out damage assessment, such as incomplete damage statistics from surveys, bias in the opinion of experts, limitations of computer models, etc. The development of fragility curves in hybrid methods is based on the combination of several damage prediction methods.

The empirical and judgement-based methods are suitable for a set of buildings whereas mechanical and hybrid methods are employed for individual building analysis.

7.4 Vulnerability Assessment of Adobe Buildings

Various approaches have been employed by researchers in the vulnerability assessment of adobe structures. Owing to the nonstandard nature of adobe materials and differences in construction practices these studies are region specific and results present significant variability.

Giovinazzi and Lagomarsino [67] developed a method for carrying out a macroseismic vulnerability assessment of building infrastructures in a given area. The seismic hazard for this method can be described using EMS-98 [55] or any other macroseismic intensity scale. In addition, the method can be employed either with the field survey data or with statistically obtained data which varies in origin and quality as compared to Europe. The method is based on determining vulnerability index (*V*) and ductility index (*Q*). The former is a measure of the ability of a building/building stock to resist lateral seismic loading. The higher is the value of *V*, the lower the building resistance and vice versa. The ductility index describes the ductility of a building/building stock and controls the rate of increase in the damage with earthquake demand level. The distribution of building damage is represented using beta distribution. For a continuous variable *x*, which ranges between *a* and *b*, the shape of distribution is controlled by the beta-distribution parameters designated as *t* and *r*. The values of *a* and *b* are taken as 0 and 6, respectively. The mean value of *x* (μ_x) can be related to μ_D through a third degree polynomial (Eq. 3).

$$\mu_x = 0.042\mu_D^3 - 0.315\mu_D^2 + 1.725\mu_D \tag{3}$$

The distribution parameters *t* and *r* are correlated with μ_D, as given in Eq. (4):

$$r = t(0.007\mu_D^3 - 0.0525\mu_D^2 + 0.287\mu_D \tag{4}$$

An analytical expression was obtained by [67], based on probability and fuzzy set theory, that relates μ_D with *V* and *Q* (Eq. 4).

$$\mu_D = \left[1 + tanh\left(\frac{I + 6.25V - 13.1}{Q}\right)\right] \tag{5}$$

A good agreement between μ_D calculated from Eq. (5), and the observed damage data was found. The values of *V*, *Q* and *t* came out to be 0.84, 2.3 and 6, respectively, for the European adobe buildings. Figure 24 illustrates vulnerability curves for adobe buildings in Europe. It is noted in Fig. 24 that the building vulnerability increases rapidly at lower intensity levels, as compared to higher levels. Figure 25 presents probability of exceedance of damage to adobe buildings versus macroseismic intensity. These are developed using the method described above for damage grades given by EMS-98. These damage grades are: D1 = slight damage; D2 = moderate damage; D3 = heavy damage; D4 = very heavy damage and D5 = total collapse. It is noted in Fig. 25 that a small percentage of buildings face the risk of collapse up to an intensity of VIII. This percentage rises above 90 % at an intensity of XII.

Demircioglu [68] employed a macroseismic approach for carrying out seismic risk studies of building typologies in Turkey. Adobe buildings were part of the building typologies and were included in this analysis. Damage states for the buildings were selected as defined by EMS-98 [55]. The vulnerability assessment methods proposed by Giovinazzi and Lagomarsino [67] and Modified KOERI [69] were employed. The latter is based on the vulnerability relationships suggested by Coburn and Spence [56]. The results from both the employed models

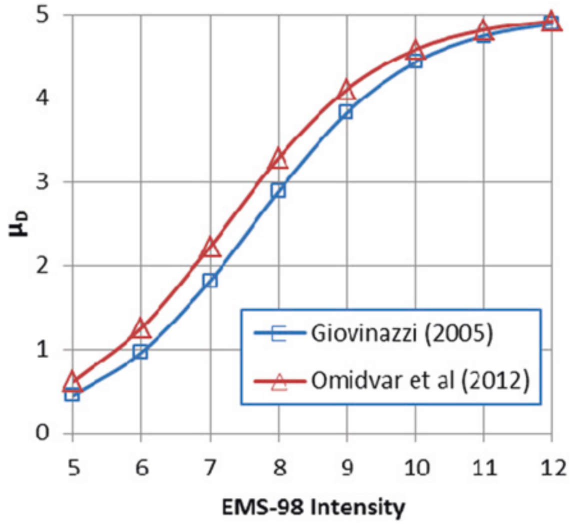


Fig. 24 Vulnerability curves for adobe buildings in Europe

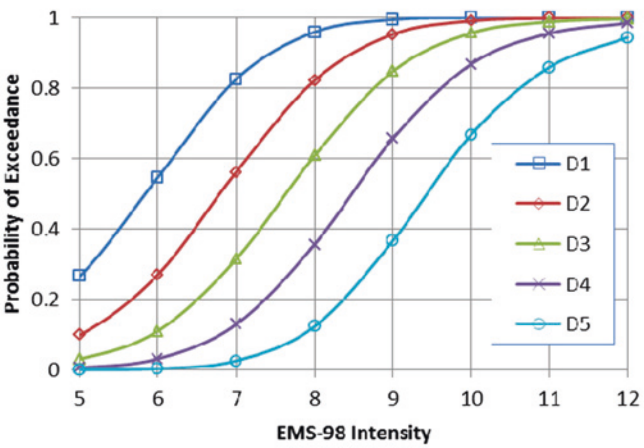


Fig. 25 Fragility curves for adobe buildings in Europe [53]

were compared with the observed earthquake damage data and a good agreement between them was found. Table 1 shows the results of *V*, *Q* and *t*, as obtained from the two models for adobe buildings in Turkey.

Omidvar et al. [70] conducted vulnerability analysis for Iranian building typologies which included adobe construction. DPM of all the building types were developed based on the observed damage data from the Bam earthquake in 2003. The macroseismic method, as suggested by Giovinazzi [67], was employed. The results indicated that adobe building damage was initiated at an earthquake intensity of VIII on EMS-98 intensity scale. The vulnerability index (V) for adobe

Table 1 Comparison of parameters for vulnerability assessment models [68]

	Modified KOERI method [69]			Giovinazzi and Lagomarsino method [67]		
Adobe building	*V*	*Q*	*t*	*V*	*Q*	*t*
Low rise	0.6	2.3	4	0.84	2.3	6
Mid rise	0.6	2.3	4	0.84	2.3	6
High rise	0.6	2.3	4	0.84	2.3	6

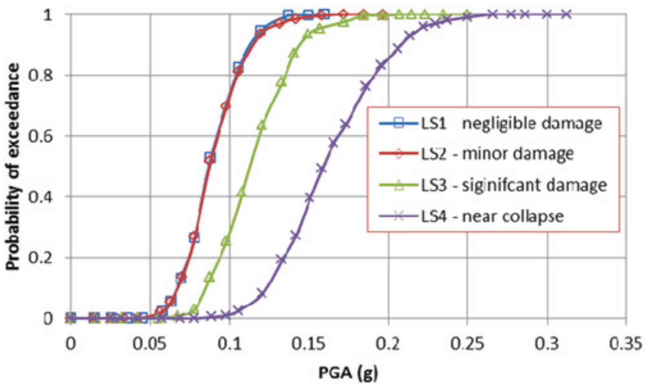


Fig. 26 Fragility curves for in-plane adobe behaviour [72]

buildings in Iran came out to be 0.9. The results co-related well with those presented by JICA [71] for adobe buildings in Tehran on the basis of Manjil earthquake in 1990.

Tarque [72] carried out a seismic risk analysis of adobe buildings in Cusco (Peru) which considered both the in-plane and out-of-plane adobe wall behaviour. Displacement response spectra of Cusco for different earthquake return periods were obtained using PSHA. These were employed to determine the demand for adobe structures. The capacities of the buildings were calculated for different limit states (LS) in terms of displacement capacity and period of vibration (mechanical displacement based procedures). The probabilities of failure for the employed return periods were calculated by comparing the capacity with the demand. Fragility curves were plotted for conditional seismic risk analysis. These relate failure probability in each limit state, as a function PGA (and its associated return period) for both in-plane and out-of-plane behaviour. For the unconditional analysis, all the ground motions with their return periods were considered for a specified time window, up to 100 years. The results of fragility curves indicated that 77 % of adobe buildings will have in-plane failure and 75 % buildings will develop wide cracks at the wall junction due to out-of-plane wall bending at a PGA of 0.18 g which is recommended by the Adobe Peruvian Code for Cusco region. Figure 26 illustrates fragility curves for in-plane behaviour at different limit states.

8 Strengthening and Retrofitting Solutions

Walls are the fundamental structural elements in earthen buildings. Earthquakes cause the sudden formation of cracks in the adobe walls at the beginning of any ground motion. Adequate seismic reinforcement solutions are needed to assure the safety of adobe construction by controlling the displacements of cracked walls. Furthermore, due to the fact that the large majority of adobe dwellings are located in developing countries, the implementation of low-cost seismic strengthening solutions using widely available materials is desirable.

Several studies to achieve this goal have been conducted [2, 27]. The main objectives of the developed strengthening or reinforcement schemes are to assure a proper connection between construction elements and to reach overall stability.

One simple and effective method for structural rehabilitation of adobe structures in general and also valuable for seismic retrofit is the injection of grouts into the cracks existing on the constructions [73]. Other traditional techniques used to repair cracks in adobe constructions can be very disturbing and intrusive when compared to grout injection. This could be, however, a non-reversible technique, which can originate durability and compatibility problems if non-suitable materials are chosen to compose the grout, particularly for earthen structures [74]. Earthen grouts could be good enough to get a restitution of the low tensile strength of earthen construction.

The improvement of the mechanical behaviour requires a fluid grout with very good penetrability and bonding properties, while durability requires the development of a microstructure as close as possible to the microstructure of the existing materials. Currently, a design methodology for grout injection of earthen constructions is under study [75], which could represent an important step forward in the repair of these structures. However, mechanical injection techniques are not yet totally developed.

Regarding seismic retrofit, grout injection must be complemented with other techniques that could increase ductility to dissipate seismic energy. From several possible solutions, the following are mentioned: cane or timber internal reinforcements, cane external reinforcement, reinforced concrete as internal reinforcement and synthetic mesh strengthening systems.

8.1 Cane or Timber Internal Reinforcements

This type of reinforcement consists of placing an internal grid, with vertical and horizontal elements, able to bond efficiently with the structure, improving its seismic performance (see example in Fig. 27). The vertical elements should be conveniently anchored to the foundation and to a ring beam on top of the walls. The spacing of the vertical or horizontal elements should be such to provide an efficient connection to the structure. Bamboo canes or eucalypt dry timber is

Fig. 27 Internal cane mesh reinforcement [77]

recommended for these reinforcements [76, 77]. It should be noted that this type of reinforcement can only be done in new constructions.

The placement of the horizontal layers should be carefully carried out, as these can become weak points, which, under seismic forces, can cause horizontal cracks. For adobe structures, in order to provide an effective bonding, mortar thickness between two rows of adobe blocks, with reinforcement in between, can become larger than desirable [78]. Laboratory tests proved that high thickness mortars correspond to lower wall masonry strength.

Full-scale shaking table tests were conducted with adobe houses using this kind of reinforcement, demonstrating a good response to save lives [79]. The model reinforced with an internal cane mesh suffered significant damage, but did not collapse. A major restraint in using this strengthening solution is the fact that cane or adequate timber is not available in all seismic regions.

8.2 Cane External Reinforcement

For repair or seismic retrofit of existing structures, an external reinforcement using a grid of canes and ropes can be a good solution. Canes are placed vertically and externally to the wall, on both sides, inside and outside. Ropes are then positioned horizontally tying the vertical canes along the walls and involving the structure. Different rows of horizontal ropes are placed along the height of the wall with a spacing of 30–40 cm. In order to connect the two grids—outside and inside grids—and thus confine the earthen structure, small extension lines are placed connecting the two grids, crossing the wall from one side to another through holes, made at each 30–40 cm. This reinforcement grid can then be covered with plaster for adequate finishing, providing at the same time more confinement to the earth structure.

Fig. 28 External cane-rope mesh reinforcement [80]

Figure 28 shows an example of this type of reinforcement applied to a real-scale model tested at the PUCP, where only part of the structure was covered with plaster.

The main limitation of this type of reinforcement is the fact that a great quantity of cane is required. As cane is not available in all regions, industrial material must be studied and tested.

8.3 Reinforced Concrete Elements as Reinforcement

This technique consists of building first the adobe walls with gaps in the corners, or connections with other walls to be filled by concrete. Steel bars are then placed and the concrete is poured in order to form a confined system with columns and collar beam. This solution is rather expensive, conducting to a high stiffness system with low ductility [78]. Furthermore, important collapses in earthen construction with reinforced concrete elements were reported, implying that this can be an inadequate reinforcement solution, though more studies on the subject are required. Figure 29 shows examples of collapses after reinforcement actions using concrete: Tarapacá Cathedral, Iquique earthquake, 2005, Chile, and San Luis de Cañete Church, Pisco earthquake, 2007, Peru. In Tarapacá Cathedral, the bending of the reinforced concrete beam destroyed completely the main adobe wall with 1.30 m of thickness. In San Luis de Cañete Church, the reinforced concrete frames changed the global behaviour of the structure, creating discontinuities, and the adobe walls overturned during the earthquake.

8.4 Synthetic Mesh Strengthening Solutions

Reinforcement solutions with synthetic meshes (geogrids) involving the walls have been studied and tested, proving its applicability, simplicity and efficiency. Figure 30 shows examples of application. In Figueiredo et al. [28] and Oliveira

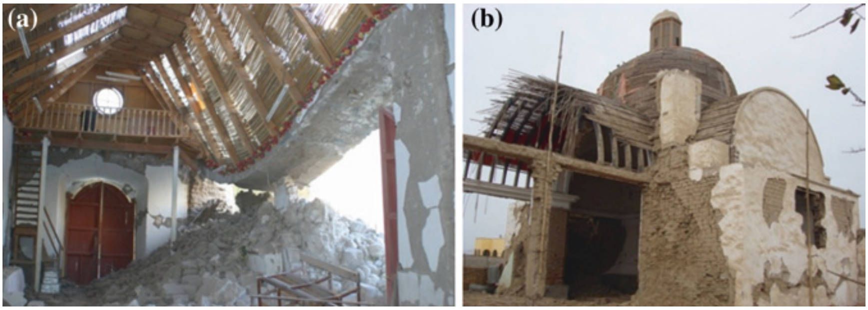


Fig. 29 Examples of collapses after reinforcement using concrete. **a** Tarapacá Cathedral, Chile, 2005 (courtesy: Chesta J.). **b** San Luis de Cañete Church, Peru, 2007 [81]

Fig. 30 Synthetic meshes used in adobe structures strengthening. **a** Double-T adobe wall [82]. **b** Adobe module [83]

et al. [82], is described a test performed in a real-scale adobe wall with and without reinforcement and the results of both tests were compared. The solution for filling the wall cracks (injection of hydraulic lime grout) combined with the strengthening solution (synthetic mesh incorporated in the plaster) proved to be very effective. Figure 31 shows the comparison between the results obtained for the original wall and the strengthened wall. The tests on the retrofitted wall demonstrated that the lateral strength increased slightly, and the ductility and the energy dissipation capacity improved significantly. The wall was able to recover its initial stiffness.

In Blondet et al. [3], several similar full-scale adobe housing models with different density and types of synthetic mesh were tested in a shaking-table. The results showed that the damage decreased as the amount of synthetic mesh placed involving the walls increased. In Vargas et al. [81], the use of geogrid in adobe constructions is extensively explained, with comprehensible details on how to cut and place the grid with the objective of improving seismic performance.

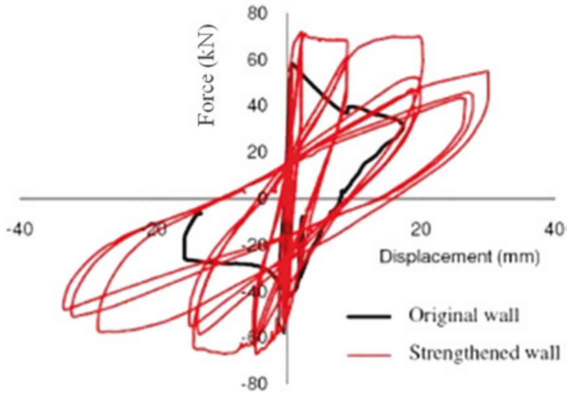


Fig. 31 Horizontal force versus lateral displacement for the original and strengthened walls [82]

The use of synthetic mesh bands involving the adobe walls and covered with mortar is also possible. The mesh is placed in horizontal and vertical strips, following a layout similar to that of beams and columns. This solution is able to provide additional strength to the structure, though the failure mode observed was brittle and dangerous. The use of cement also makes this an expensive solution.

9 Final Comments

The adobe bricks are basically composed of farm soil mixed with other materials, like straw or lime, depending on the soil characteristics. The bricks are sun-dried and the building construction process passed from generation to generation, in a learning by doing process. In comparison to other building materials, as reinforced concrete, the adobe is costless, for this reason it is massively used by families with low-incomes, without paying adequate attention on the structural safety and reinforcement issues [84]. In many regions of the World, vernacular buildings have thickness of around 0.30 m, which may influences on the stability of walls due to their pronounced slenderness.

Adobe is classified as a brittle material with a fragile behaviour, in particular when subjected to horizontal forces, like those induced by earthquakes. Adobe constructions are heavy, brittle and during earthquakes may attract large inertia forces that lead to the collapse of the adobe constructions. Survey of damage and experimental test results on adobe buildings subjected to earthquake demands reveals that one of the most common failure mechanisms in these constructions is the overturning on the walls out-of-plane. For this reason, a simple improvement of the diaphragm (roof) may stabilize and promote the “box behaviour” of the construction. In recent years, some solutions have been proposed and studied to improve the seismic behaviour of adobe constructions, and some low-cost and easy to apply solutions have been implemented in adobe constructions as in Pisco (Peru).

The results of tests and numerical models developed and calibrated allow for a better understanding of the behaviour of adobes constructions under different loading conditions, which may help in designing economical and sustainable rehabilitation and strengthening solutions for the existing constructions.

The rehabilitation and strengthening solutions studied by different research groups showed to be efficient, as observed in their test results. It was concluded that these techniques are affordable and easy to apply techniques to improve the seismic behaviour of adobe masonry constructions.

The information given in this chapter pretends to summarize the actual knowledge and research outputs on the structural behaviour of adobe buildings. It is clear the need for further research in this field, and particularly for other types of constructions, as monuments, to which the strengthening should provide seismic capacity, but using non-intrusive solutions.

References

1. Houben, H., Guillard, H.: Earth Construction: a Comprehensive Guide. Practical Action, London (1994)
2. Memari, A.M., Kauffman, A.: Review of existing seismic retrofit methodologies for adobe dwellings and introduction of a new concept. In: Proceedings of SismoAdobe2005, Pontificia Universidad Católica del Perú, 15, Lima, Peru. (2005)
3. Blondet, M., Vargas, J., Velásquez, J., Tarque, N.: Experimental study of synthetic mesh reinforcement of historical adobe buildings. In: Lourenço, P.B., Roca, P., Modena, C., Agrawal. S. (eds) Proceedings of Structural Analysis of Historical Constructions, pp. 1–8, New Delhi, India (2006)
4. Dowling, D.: Adobe housing in El Salvador: Earthquake performance and seismic improvement. In: Rose, I., Bommer, J.J., López, D.L., Carr, M.J., Major, J.J. (eds) Geological Society of America Special Papers, 281–300 (2004)
5. De Sensi, B.: Terracruda, la diffusione dell'architettura di terra. www.terracruda.com/architetturadiffusione.htm (2003)
6. Lowman, P.D., Montgomery, B.C.: Preliminary determination of epicenters of 358,214 events between 1963 and 1998, (2008). http://denali.gsfc.nasa.gov/dtam/seismic/. Accessed in June 2011
7. Fratini, F., Pecchioni, E., Rovero, L., Tonietti, U.: The earth in the architecture of the historical centre of Lamezia Terme (Italy): Characterization for restoration. Constr. Build. Mater. **53**, 509–516 (2011)
8. Fernandes, M., Portugal, M.V.: Atlântico versus Portugal Mediterrâneo: Tipologias arquitectónicas em terra. In: International Conference TerraBrasil 2006, Ouro Preto, Minas Gerais, Brazil (2006)
9. Varum, H., Figueiredo, A., Silveira, D., Martins, T., Costa, A.: Outputs from the research developed at the University of Aveiro regarding the mechanical characterization of existing adobe constructions in Portugal—Informes de la Construcción, doi: 10.3989/ic.10.016, July–Sept 2011, **63**(523), 127–142 (2011)
10. Tavares, A., Costa, A., Varum, H.: Adobe and modernism in Ílhavo, Portugal. Int. J. Architect. Heritage, Taylor & Francis, ISSN 1558–3058, Paper reference ID UARC-2011-0357.R1, **6**(5), 525–541, doi: 10.1080/15583058.2011.590267 (2012)
11. Sandrolini, F., Franzoni, E., Varum, H., Niezabitowska, E.: Materials and technologies in Art Nouveau architecture: Façade decoration cases in Italy, Portugal and Poland for a consistent

restoration—Informes de la Construcción, Instituto de Ciencias de la Construcción Eduardo Torroja (IETcc), ISSN 0020-0883, doi 10.3989/ic.10.053, **63**(524), 5–11 (2011)
12. Dell'Acqua, A.C., Franzoni, E., Sandrolini, F., Varum, H.: Materials and techniques of Art Nouveau architecture in Italy and Portugal: A first insight for an European route to consistent restoration—International Journal for Restoration of Buildings and Monuments, Aedificatio Verlag Publishers, ISSN 1864–7251 (print-version); ISSN 1864-7022 (online-version), **15**(2), 129–143 (2009)
13. Blondet, M., Villa-García, G., Brzev S.: Earthquake-resistant construction of adobe buildings: In: Greene, M. (ed) A Tutorial, Report, EERI/IAEE world Housing Enciclopedy, Okland, California, USA (2003)
14. INEI (National Institution for Statistics and Informatic): Census 2007: XI de Población y VI de Vivienda" Report. National Institute of Statistics and Informatics, Lima (2008)
15. Mousavi, S.E., Khosravifar, A., Bakhshi, A., Taheri, A., Bozorgnia, Y.: Structural typology of traditional houses in iran based on their seismic behaviour.In: Proceedings of 8th U.S. National Conference on Earthquake Engineering, (2006)
16. Baglioni, E., Fratini, F., Rovero, L.: The material utilised in the earthen buildings sited in the Drâa Valley (Morocco): Mineral and mechanical characteristics. 6ATP, 9SIACOT, (2010)
17. Ruano, A., Costa, A.G., Varum, H.: Study of the common pathologies in composite adobe and reinforced concrete constructions. ASCE's J. Perform. Constr. Facil. Am. Soc. Civil Eng. doi 10.1061/(ASCE)CF.1943-5509.0000200, **26**(4), 389–401 (2012)
18. Blondet, M., Vargas, J., Tarque, N.: Observed behaviour of earthen structures during the Pisco earthquake (Peru). In: Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China (2008)
19. Webster, F., Tolles. L.: Earthquake damage to historic and older adobe buildings during the 1994 Northridge, California Earthquake. In: Proceedings of 12th World Conference on Earthquake Engineering, Auckland, New Zealand (2000)
20. ASTM E519/E519 M–10: Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages. ASTM International, West Conshohocken (2010)
21. EN 1052–1: Methods of test for masonry—Part 1: Determination of compressive strength. European Committee for Standardization (CEN), Brussels (1998)
22. Martins, T., Varum, H.: Adobe's Mechanical Characterization in Ancient Constructions: The Case of Aveiro's Region—Materials Science Forum, Trans Tech Publications, Switzerland, ISSN 0255-5476, vol. 514–516, pp. 1571–1575 (2006)
23. Silveira, D., Varum, H., Costa, A.: Influence of the testing procedures in the mechanical characterization of adobe bricks. Constr. Build. Mater. J. doi 10.1016/j.conbuildmat.2012.11.058, **40**, 719–728 (2013)
24. Silveira, D., Varum, H., Costa, A., Martins, T., Pereira, H., Almeida, J.: Mechanical properties of adobe bricks in ancient constructions—Construction & Building Materials, Elsevier, Manuscript reference CONBUILDMAT-D-11-00604, doi 10.1016/j.conbuildmat.2011.08.046, **28**, 36–44 (2012)
25. Blondet, M., Vargas, J.: Investigación sobre vivienda rural. Report, Division of Civil Engineering, Pontificia Universidad Católica del Perú, Lima, Peru (1978)
26. Vargas, J., Ottazzi, G.: Investigaciones en adobe. Report, Division of Civil Engineering, Pontificia Universidad Católica del Perú, Lima, (1981)
27. Blondet, M., Madueño, I., Torrealva, D., Villa-García, G., Ginocchio, F.: Using industrial materials for the construction of safe adobe houses in seismic areas. In: Proceedings of Earth Build 2005 Conference, Sydney, Australia, (2005)
28. Figueiredo, A., Varum, H., Costa, A., Silveira, D., Oliveira, C.: Seismic retrofitting solution of an adobe masonry wall. Mater. Struct. RILEM, ISSN 1359-5997, doi 10.1617/s11527-012-9895-1, **46**(1–2), 203–219 (2013)
29. Tareco, H., Grangeia, C., Varum, H., Senos-Matias, M.: A high resolution GPR experiment to characterize the internal structure of a damaged adobe wall. EAGE First Break **27**(8), 79–84 (2009)

30. Antunes, P., Lima, H., Varum, H., André, P.: Optical fiber sensors for static and dynamic health monitoring of civil engineering infrastructures abode wall case study. Measurement **45**, 1695–1705 (2012)
31. Stavridis, A., Shing, P.B.: Finite Element Modeling of Nonlinear Behavior of Masonry-Infilled RC Frames. J. Struct. Eng. ASCE 2010, **136**(3), 285–296 (2010)
32. Kappos, A.J., Penelis, G.G., Drakopoulos, C.G.: Evaluation of simplified models for lateral load analysis of unreinforced masonry buildings. J. Struct. Eng. **128**(7), 890 (2002)
33. Lourenço, P.B.: Computational strategies for masonry structures. Ph.D. Thesis, Delft University, Delft, The Netherlands (1996)
34. Page, A.W.: Finite element model for masonry. J. Struct. Eng. **104**(8), 1267–1285 (1978)
35. Roca, P., Cervera, M., Gariup, G., Pela, L.: Structural analysis of masonry historical constructions. Classical and advanced approaches. Arch. Comput. Methods Eng **17**(3), 299–325 (2010)
36. Calderini, C., Cattari, S., Lagomarsino, S.: In plane seismic response of unreinforced masonry walls: comparison between detailed and equivalent frame models. In: Papadrakakis, M., Lagaros, N.D., Fragiadakis, M. (eds) ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Rhodes, Greece, (2009)
37. Gambarotta, L., Lagomarsino, S.: Damage models for the seismic response of brick masonry shear walls. Part I: The mortar joint and its applications. Earthquake Eng. Struct. Dynamics **26**(4), 423–439 (1997)
38. Gambarotta, L., Lagomarsino, S.: Damage models for the seismic response of brick masonry shear walls. Part II: The continuum model and its applications. Earthquake Eng. Struct. Dyn. **26**(4), 441–462 (1997)
39. Lagomarsino, S., Galasco, A., Penna A.: Non-linear macro-element dynamic analysis of masonry buildings. In: Proceedings of ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Rethymno, Crete, Greece, (2007)
40. Magenes, G., Della, F.A.: Simplified non-linear seismic analysis of masonry buildings. In: Proceedings of Fifth International Masonry Conference, British Masonry Society, London, England, (1998)
41. Ali, S.S., Page, A.W.: Finite element model for masonry subjected to concentrated loads. J. Struct. Eng. **114**(8), 1761 (1987)
42. Cao, Z., Watanabe, H.: Earthquake response predication and retrofitting techniques of adobe structures. In: Proceedings of 13th World Conference on Earthquake Engineering, Vancouver, Canada, (2004)
43. Furukawa, A., Ohta, Y.: Failure process of masonry buildings during earthquake and associated casualty risk evaluation. Nat. Hazards, **49**, 25–51 (2009)
44. Rots, J.G.: Numerical simulation of cracking in structural masonry. Heron **36**(2), 49–63 (1991)
45. Arya, S.K., Hegemier, G.A.: On nonlinear response prediction of concrete masonry assemblies. In: Proceedings of North American Masonry Conference, Boulder, Colorado, USA, pp 19.1–19.24 (1978)
46. Lotfi, H.R., Shing, P.B.: Interface Model Applied to fracture of masonry Structures. ASCE **120**(1), 63–80 (1994)
47. Lourenço, P.B., Rots, J.G.: Multisurface interface model for analysis of masonry structures. J. Eng. Mech. **123**(7), 660 (1997)
48. Cundall, P.A.: A computer model for simulating progressive large scale movements in blocky rock systems. In: Proceedings of Symposium on Rock Fracture (ISRM), Nancy, France, (1971)
49. Alexandris, A., Protopapa, E., Psycharis, I.: Collapse mechanisms of masonry buildings derived by distinct element method. In: Proceedings of 13th World Conference on Earthquake Engineering, (2004)
50. Tarque, N.: Numerical modelling of the seismic behaviour of adobe buildings. PhD thesis. Universitá degli Studi di Pavia, Istituto Universitario di Studi Superiori. Pavia, Italy, (2011)

51. Bommer, J.J., Scherbaum, F., Bungum, H., Cotton, F., Sabetta, F., Abrahamson, N.A.: On the use of logic trees for ground-motion prediction equations in seismic hazard analysis. Bull. Seismol. Soc. Am. **95**(2), 377–389 (2005)
52. Foerster, E., Krien, Y., Dandoulaki, M., Priest, S., Tapsell, S., Delmonaco, G., Margottini, C., Bonadonna, C.: Methodologies to assess vulnerability of structural systems, pp. 1–139. Seventh Framework Programme, European Commission (2009)
53. Giovinazzi, S.: The Vulnerability Assessment and the Damage Scenario in Seismic Risk Analysis. PhD Thesis, University of Florence and Technical University of Braunschweig, (2005)
54. Kramer, S.: Geotechnical Earthquake Engineering. Prentice Hall International Series, Ohio (1996)
55. Grünthal, G.: European Macroseicmic Scale (EMS-98), Cahiers du Centre Européen de Géodynamique et de Séismologie 15. Centre Européen de Géodynamique et de Séismologie, Luxembourg (1998)
56. Coburn, A., Spence, R.: Earthquake Protection. Wiley, USA (2002)
57. Jozefiak, S.: Fragility Curves for Simple Retrofitted Structures. CM-4 Consequence, (2005)
58. Calvi, G.M., Pinho, R., Magenes, G., Bommer, J.J., Restrepo-Velez, L.F., Crowley, H.: Development of seismic vulnerability assessment methodologies over the past 30 years. J. Earthquake Technol. paper no 472, **43**(3), 75–104 (2006)
59. Whitman, R.V., Reed, J.W., Hong, S.T.: Earthquake damage probability matrices. In: Procedings of 5th European Conference on Earthquake Engineering, Rome, 25–31, (1973)
60. Benedetti, D., Sulla, Petrini V.: Vulnerabilità di Edifici in Muratura: Proposta di un Metodo di Valutazione. L'industria delle Costruzioni **149**(1), 66–74 (1984)
61. Lagomarsino, S., Giovinazzi, S.: Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings. Bull. Earthq. Eng. **4**, 445–463 (2006)
62. Spence, R., Coburn, A.W., Pomonis, A.: correlation of ground motion with building damage: the definition of a new damage-based seismic intensity scale. In: Proceedings of the Tenth World Conference on Earthquake Engineering, Madrid, Spain, vol. 1, pp. 551–556 (1992)
63. Applied Technology Council (ATC): Earthquake damage evaluation data for California, Applied Technology Council, ATC-13. Redwood, California (1985)
64. Barbat, A.H., Moya, F.Y., Canas, J.A.: Damage scenarios simulation for seismic risk assessment in urban zones. Earthquake Spectra **12**(3), 371–394 (1996)
65. Kappos, A.J., Panagopoulos, G., Panagiotopoulos, C., Penelis, G.: A hybrid method for the vulnerability assessment of R/C and URM buildings. Bull. Earthquake Eng. **4**, 391–413 (2006)
66. Vicente, R., Parodi, S., Lagomarsino, S., Varum, H., Silva, J., Mendes, A.R.: Seismic vulnerability and risk assessment: Case study of the historic city centre of Coimbra, Portugal. Bulletin of Earthquake Engineering, Springer, Manuscript Ref. BEEE325R1, doi 10.1007/s10518-010-9233-3, vol. 9, 1067–1096 (2011)
67. Giovinazzi, S., Lagomarsino, S.: Fuzzy-Random Approach for a Seismic Vulnerability Model. In: Proceedings of ICOSSAR, Rome, Italy (2005)
68. Demircioglu, M.B.: Earthquake Hazard and Risk Assessment for Turkey. PhD Thesis. Bogaziçi University, Turkey, (2010)
69. DEE-KOERI.: Earthquake Risk Assessment for the Istanbul Metropolitan Area. Report prepared by Department of Earthquake Engineering-Kandilli Observatory and Earthquake Research Institute, Bogazici University Press, Istanbul (2003)
70. Omidvar, B., Gatmini, B., Derakhshan, S.: Experimental vulnerability curves for the residential buildings of Iran. J. Nat. Hazards **60**(2), 345–365 (2012)
71. JICA.: The study on seismic microzoning of the greater Tehran area in the Islamic Republic of Iran, Final report (2000)
72. Tarque, N., Crowley, H., Varum, H., Pinho, R.: Displacement-based fragility curves for seismic assessment of adobe buildings in Cusco Peru. Earthquake Spectra J. **28**(2), 759–794 (2012)
73. Silva. R.A., Schueremans. L., Oliveira. D.V.: Grouting as a repair/strengthening solution for earth constructions. In: Proceedings of the 1st WTA International PhD Symposium, WTA publications, Leuven, pp. 517–535 (2009)

74. The Getty Conservation Institute, Pontificia Universidad Católica del Perú: Interdisciplinary Experts Meeting on Grouting Repairs for Large-scale Structural Cracks in Historic Earthen Buildings in Seismic Areas. Peru (2007)
75. Vargas-Neumann, J., Blondet, M., Ginocchio, F., Morales, K., Iwaki, C.: Uso de grouts de barro líquido para reparar fisuras estructurales en muros históricos de adobe. V Congresso de Tierra en Cuenca de Campos, Valladolid (2008)
76. NTE E.080: Norma Técnica de Edificación. "Adobe Peruvian Code". In: Spanish, MTC/SENCICO. Peru (2000)
77. Blondet, M., Vargas, J., Tarque, N., Iwaki, C.: Seismic resistant earthen construction: the contemporary experience at the Pontificia Universidad Católica del Perú. J. Informes de la Construcción **63**(523), 41–50 (2011)
78. Minke, G.: Manual de construcción para viviendas antisísmicas de tierra. Universidad de Kassel, Alemania (2001)
79. Ottazzi, G., Yep, J., Blondet, M., Villa-Garcia, G., Ginocchio, J.: Shaking table tests of improved adobe masonry houses. In: Proceedings of Ninth World Conference on Earthquake Engineering. Japan (1988)
80. Torrealva, D., Acero, J.: Reinforcing adobe buildings with exterior compatible mesh. In SismoAdobe 2005: Architecture, Construction and Conservation of Earthen Buildings in Seismic Areas, Lima, Pontificia Universidad Católica del Perú, Lima, Peru, May 16-19 2005 [CD], ed. Marcial Blondet (2005) http://www.pucp.edu.pe/eventos/ SismoAdobe2005
81. Vargas-Neumann, J., Torrealva, D., Blondet, M.: Construcción de casas saludables y sismorresistentes de adobe reforzado con geomallas. Fondo Editorial, PUCP (2007)
82. Oliveira, C., Varum, H., Figueiredo, A., Silveira, D., Costa, A.: Experimental tests for seismic assessment and strengthening of adobe structures. 14th ECEE (2010)
83. Blondet, M., Aguillar, R.: Seismic Protection of Earthen Buildings. Conferencia Internacional en Ingeniería Sísmica, Peru (2007)
84. North, G.: Waitakere City Council's Sustainable Home Guidelines—Earth building; Waitakere City Council. http://www.waitakere.govt.nz/abtcit/ec/bldsus/pdf/materials/earthbuilding.pdf (2008)

http://www.springer.com/978-3-642-39685-4

Structural Rehabilitation of Old Buildings
Costa, A.; Guedes, J.M.; Varum, H. (Eds.)
2014, VIII, 348 p. 224 illus., 147 illus. in color.,
Hardcover
ISBN: 978-3-642-39685-4