EXPERIMENTAL OUT-OF-PLANE SEISMIC RESPONSE OF FULL-SCALE EARTHEN WALLS IN HISTORIC CONSTRUCTION

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Abstract

Earthen construction has been part of human civilization, accounting for most sites in the World Heritage's List and providing housing to more than one third of the world's population. While inexpensive, widely available, sustainable, and relatively simple to build, earthen structures are also massive, brittle, and critically vulnerable to catastrophic damage and/or collapse even under low levels of ground shaking. An extensive multi-year experimental program was conducted at the Universidad de los Andes in Bogota, Colombia with the purpose of investigating the seismic response of thick earthen walls—characteristic of historic construction in South America and other parts of the world—and evaluating a wide array of retrofitting alternatives to improve their seismic behavior. This paper focuses on the results of fourteen full-scale adobe and rammed earth (RE) walls under out-of-plane actions. Ten of the specimens consisted of adobe and RE corner walls with flanges of 3.00 m and 1.8 m long, 3.45 m height, and 0.6 m thickness subjected to four levels of increasing seismic intensity. The remaining specimens consisted of two adobe walls with dimensions of 2.35 m × 1.25 m × 0.6 m under cyclic loading and four adobe walls with dimensions of 2.7 m × 1.8 m × 0.4 m under quasi-static loading. Retrofitting alternatives included steel and synthetic meshes, steel plates, and a combination of timber planks and vertical post-tensioning. It was found that meshes are efficient in preventing collapse and increasing displacement capacity after cracking of the earthen walls takes place, but they may not be feasible for heritage construction due to the occurrence of permanent drifts in excess of 1%. Steel plates, on the other hand, stiffened the walls and increased the cyclic energy dissipation capacity of adobe specimens by more than 100%, while delaying the occurrence of damage to drift ratios at least twice as large as that for an unretrofitted counterpart. Timber planks combined with vertical tensors increased the out-of-plane static capacity of adobe walls by a factor of more than 2.5 and rendered much smaller permanent deformations under ground shaking as compared to walls retrofitted with meshes. Finally, steel plates significantly improved the behavior of RE corner walls, preventing collapse at extreme ground motions while rendering small residual drifts. The use of any of the retrofitting techniques evaluated in the experimental program allowed earthen walls to sustain ground motions well in excess the design earthquake. By contrast, unretrofitted counterpart specimens failed even under a limited safety earthquake with probability of exceedance of 20% in 50 years.

Keywords: earthen walls; cyclic tests; shake table tests; dynamic; shear walls; out-of-plane seismic response.

1. Introduction

Although construction and material technologies have evolved at great speed in the last few decades, earthen construction still provides housing for nearly half of the world's population [1]. Earth-based materials such as adobe and rammed earth (RE) are attractive because of their low cost, wide availability, sustainability, simplicity of construction, and significant thermal and acoustic isolation. Adobe construction is usually made of unbaked earthen brick units running bond and joined by a mud-based mortar usually made from native soils [2, 3]. RE construction, on the other hand, consist of compacted layers of moist earth built inside a movable formwork [4, 5].

Due to its worldwide use, earthen construction is also predominant in regions of high seismic risk. Unfortunately, the brittle and massive nature of components along with the lack of detailing intrinsic to non-engineered construction have rendered adobe and RE structures especially vulnerable to earthquake damage. Testimonies of this vulnerabilities are sadly illustrated by many seismic events over the years such as the Peru 2007 earthquake which killed over 600 people and destroyed more than 75,000 houses [6] and the Ludian 2014 earthquake that caused severely damage or collapse to 80,900 earthen houses and killed 617 people [7]. In recognition to the seismic risk of earthen construction, efforts to preserve heritage buildings in Colombia have taken place during the last decades, but documentation of retrofitting schemes has been both scarce and scattered. In response, the Ministry of Culture the Bogota District Institute for Cultural Heritage (IDPC) partnered with the Colombian Association for Earthquake Engineering (AIS) and the Research Center on Materials and Civil Infrastructure (CIMOC) at Universidad de los Andes in Bogota to investigate alternative solutions to protect historic and older buildings in Colombia. The partnership led to the execution of an extensive multi-phase experimental program to evaluate the seismic behavior of unretrofitted and retrofitted thick earthen walls—characteristic of the local historical construction. This paper presents relevant results for a subset of specimens subjected to out-of-plane loading along with the evaluation of their response after retrofitting with engineered and non-engineered materials.

2. Material characterization tests

To best represent the seismic response of local heritage construction in Colombia, the characterization and full-scale wall specimens were built using raw materials that were recovered from century-old dwellings. Local soil from a demolished heritage building complex were first cleaned from coarse aggregates and organic materials and then used to make mortar for the adobe walls and earth for the construction of the RE wall specimens. Fortunately, the number of recovered adobe brick units (with typical dimensions of 70 × 140 × 280 mm) were sufficient for the construction of both material characterization assemblies and full-scale adobe specimens.

The material characterization program included soil physical properties (such as gradation, Atterberg's limits, and modified compaction tests), compression tests of 33 adobe units and bending tests of seven adobe units (for the determination of modulus of rupture). Three types of material assembly characterization tests were also performed as illustrated in Fig. 1, including: a) compression tests of six adobe and five RE blocks each with dimensions of 140 × 280 mm in cross-section by 320 mm in length; b) diagonal tension tests of ten adobe wall panels of 700 × 700 mm by 280 mm thickness; c) diagonal tension tests of thirteen RE wall panels of 1000 × 1000 mm by 400 mm thickness. The median compressive strength of individual adobe units was found to be 2800 kN/m^2 (coefficient of variation, CV = 0.3), which was much higher than that of the adobe block assemblies (1350 kN/m^2 with CV = 0.14). The median modulus of rupture of individual adobe blocks was found to be 480 kN/m^2 (CV = 0.36), while the median diagonal tension capacity of adobe wallettes was 27 kN/m^2 (CV = 0.28). The RE assembly characterization tests rendered median values of compression = 1100 kN/m^2 (CV = 0.08) and diagonal tension =31 kN/m^2 (CV = 0.38). Cube specimens made of mortar typically used in the construction of adobe walls (with lime and sand in a 1:2 proportion) had compression and tensile strengths of 2500 and 147 kN/m^2, respectively.

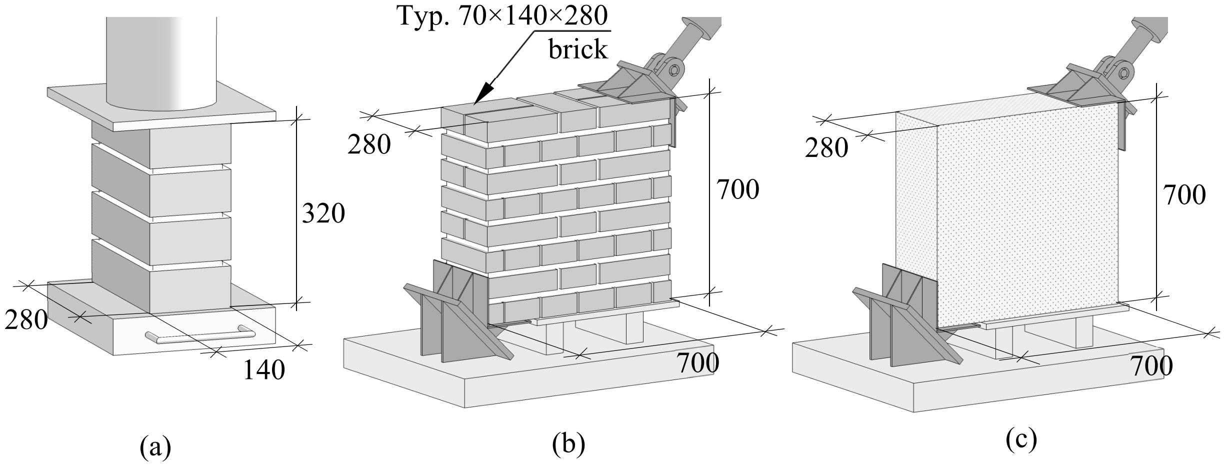


Fig. 1 – Material characterization assembly tests: a) compression (adobe & RE); b) diagonal tension of adobe; c) diagonal tension of RE.

3. Overturning tests of adobe walls retrofitted with timber planks

Four specimens with overall dimensions of 2700 mm long by 1800 mm tall and 400 mm thickness were tested under monotonically increasing uniform out-of-plane loading. The geometry of the specimens included boundary elements of 500 × 400 mm on both ends to represent the effect of orthogonal walls (Fig. 2a). Axial load was applied using a hydraulic ram and four steel rods. Two of the specimens were unretrofitted while the other two were retrofitted with timber planks of 150 × 20 mm cross-section interconnected on both faces of the wall. More specific details about the retrofitting process are presented by Reyes et al. [8].

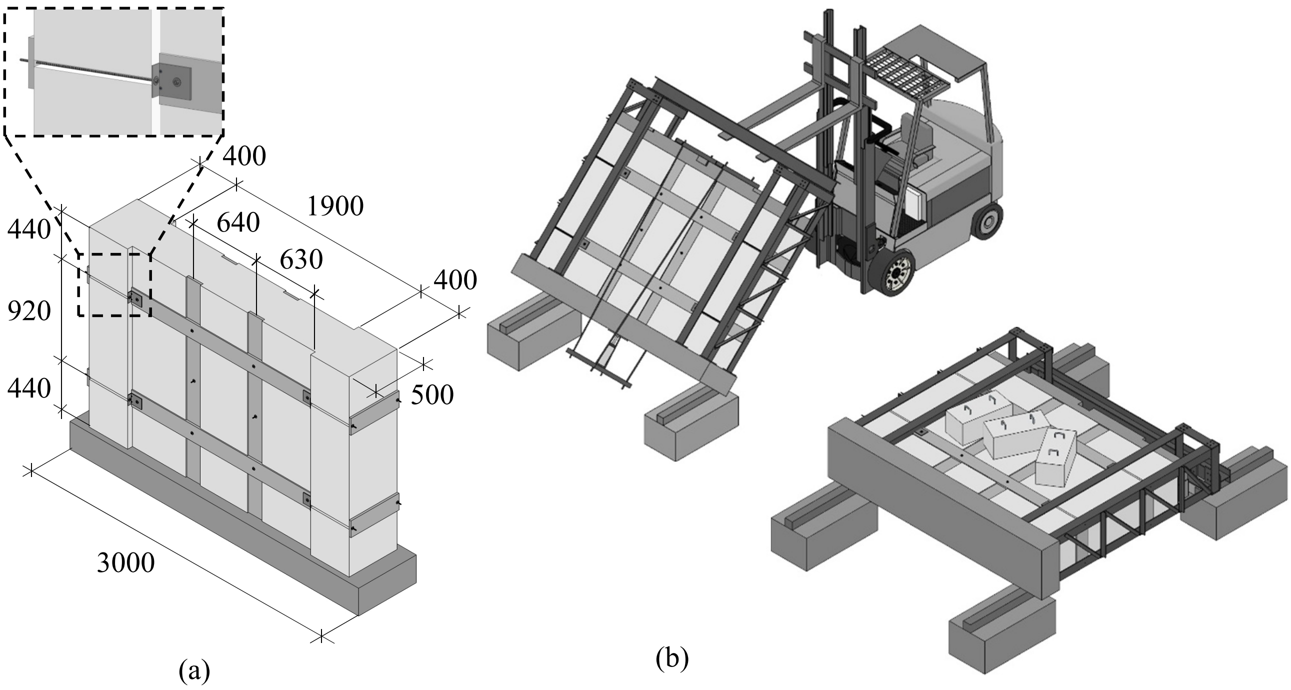


Fig. 2 – Overturning tests for adobe walls retrofitted with timber planks: (a) specimen geometry; (b) testing setup.

Each specimen was inside of a steel frame bolted to a concrete beam at the base. The boundary elements of the wall were directly supported by the steel frame, which in turn was slowly overturned by means of a forklift (Fig. 2b). The recorded inclination angle was used to estimate the out-of-plane acceleration capacity and the results are summarized in Table 1. Because the retrofitted specimens did not collapse even when the walls were fully horizontal (90-degree rotation), the axial load was then removed, and the specimens were further loaded out-of-plane by means of uniformly distributed blocks. Remarkably, the use of timber planks increased the out-of-plane capacity of the walls by at least 250% even in absence of axial compression. Limited by the available test setup, no collapse of the retrofitted specimen could be reached and only veneer peel off was observed at much higher levels of out-of-plane loading.

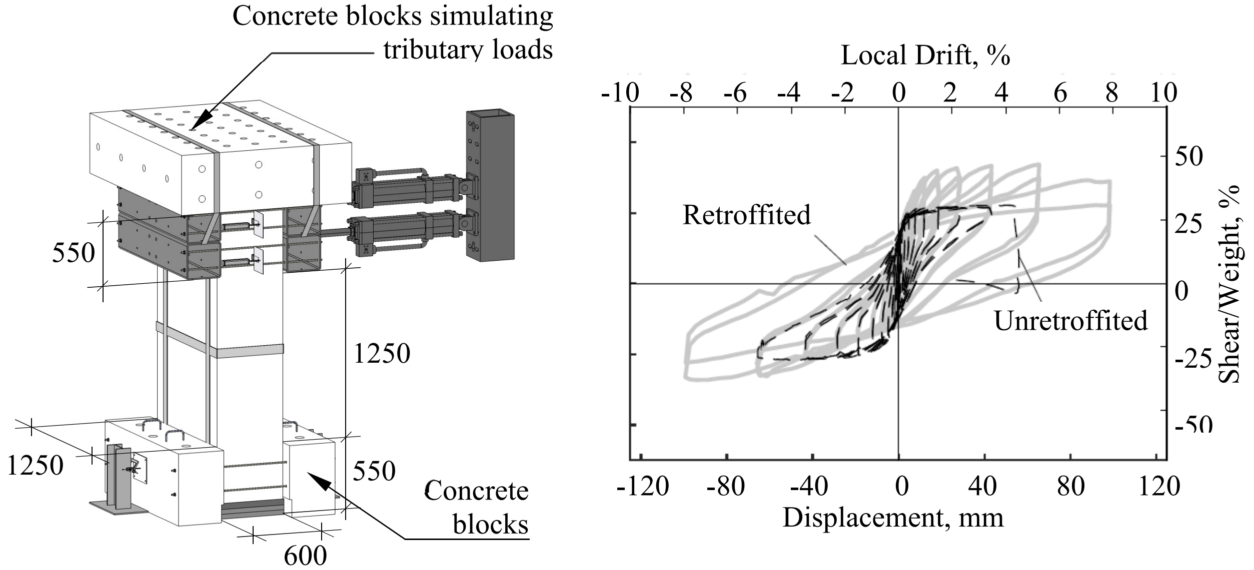
Table 1 – Results from overturning tests.

Specimen	Axial load, kN	Total out-of-plane load, kN	Final state	Maximum out-of-plane acceleration / g
unretrofitted	19.6	22.5	Collapse	0.91
unretrofitted	49.0	23.3	Collapse	0.95
retrofitted	0.00	55.5	Veneer peel off	2.30
retrofitted	0.00	55.5	Veneer peel off	2.30

4. Cyclic tests of adobe wall piers retrofitted with steel plates

Two test specimens of dimensions 1.25 m long by 2.35 m tall and 0.6 m thick were tested to study the out-of-plane cyclic behavior of wall piers between windows of typical heritage construction. To reproduce the boundary conditions at the ends of a wall pier (above and below the opening in the prototype), the upper and lower 550 mm of the test specimen was restrained laterally using steel and concrete elements as depicted in Fig. 3a. Vertical load was applied using a 15 kN concrete block to represent the tributary roof weight. One specimen was unretrofitted and the other specimen was retrofitted using 101.6 mm wide by 6.35 mm thick A36 steel plates in a grid pattern. The steel plates were placed on both sides of the wall pier and connected through 9.5mm-diameter steel rods spaced every 600 mm. The selected steel plate layout was compliant with previously proposed rehabilitation guidelines where the maximum vertical or horizontal distance between straps was established to be 1200 mm [9]. Tests were displacement-controlled using a protocol defined in FEMA 461 [10] and consisting of two cycles at increasing levels of imposed displacement at the rotation-restrained top portion of the wall pier.

Fig. 3(b) shows the measured out-of-plane force-deformation response for the unretrofitted and the retrofitted wall piers. Clearly the retrofitted specimen exhibited a greater drift capacity without brittle strength degradation and low pinching behavior even for drift ratios exceeding 5%. The marked contrast between the hysteresis loops is confirmed by the fact that the retrofitted adobe specimen had an energy dissipation capacity of more than twice that of the unretrofitted wall pier. It is also interesting to mention that the drift demand associated to the appearance of fine cracks was 2.3% for the unretrofitted specimen and 5.3% for the retrofitted pier. Similarly, the drift corresponding to near collapse increased from 5.3% to 8.0% with the implementation of the retrofitting technique.



(a) (b)

Fig. 3 – Cyclic tests for adobe walls retrofitted with steel plates: (a) specimen setup; (b) cyclic response.

5. Shake table tests of corner walls with alternative retrofitting schemes

This investigation focused on studying the seismic response of corner walls in heritage earthen construction. A literature review revealed that a very limited amount of experimental results was available for such geometrical configuration, especially for full-scale specimens with thick walls. Eight specimens were tested in the laboratory to investigate the seismic response of thick corner walls with and without implementation of alternative retrofitting techniques.

5.1 Specimen description

All eight specimens had L-shaped plan dimensions of 3000 × 1800 mm, height of 3450 mm, and thickness of 600 mm. As illustrated in Fig. 4, the specimens were intended to represent the corner portion between the main door and a side window of a typical heritage construction. To model gravity effects from roof loads, every corner wall supported concrete blocks at the top to produce an approximately uniform axial load that produced an average mid-height compressive stress of 40 kN/m^2 for the adobe walls and 44 kN/m^2 for the RE walls (the difference is due to the different unit weights of the two materials). Bolted to the shake table, the wall foundation was a U-shape reinforced concrete beam intended to replicate the confinement of a rock plinth in the prototype. To reproduce the continuity effect of an orthogonal wall, the shorter leg of the test specimen was restrained using a wide flange steel beam anchored to the foundation.

5.2 Retrofitting alternatives

The following retrofitting schemes were evaluated with the dynamic tests of corner walls: (a) timber planks combined with post-tensioning steel rods, (b) steel versus synthetic meshes, (c) steel plates.

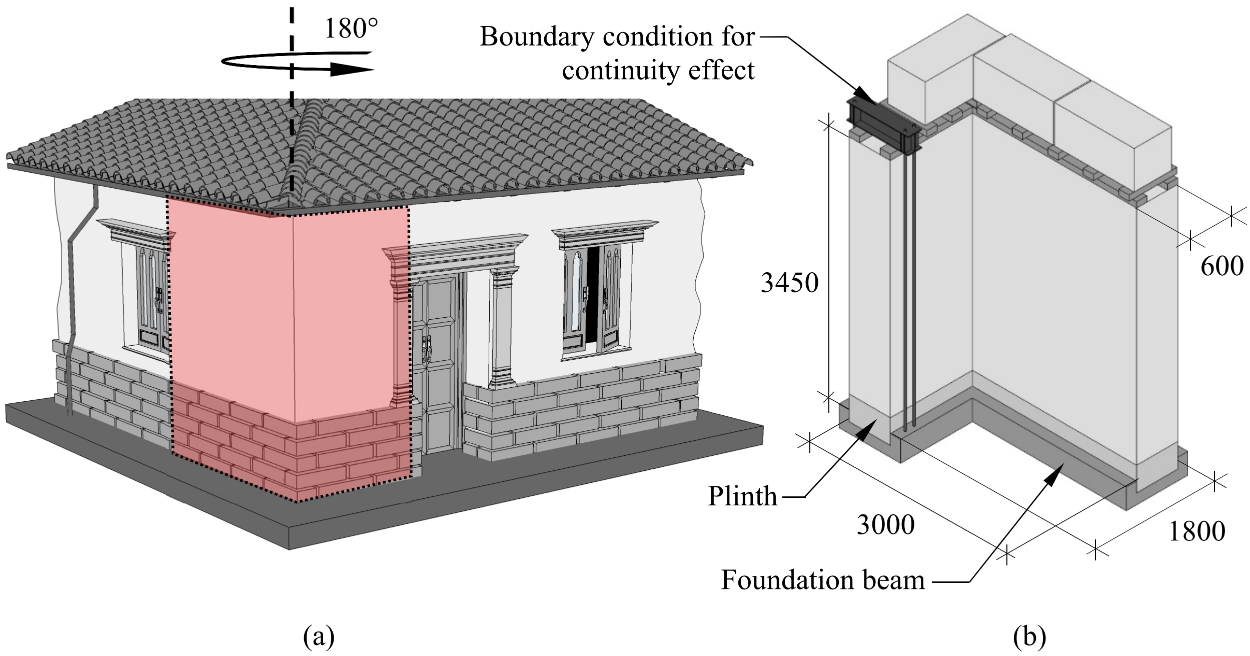


Fig. 4 – Typical earthen heritage construction and overall geometry of corner earthen wall tested in the laboratory: a) prototype component; b) test specimen.

5.2.1 Corner adobe walls retrofitted with timber planks and post-tensioning rods

Three adobe walls were simultaneously tested on the shake table. One specimen consisted of an unretrofitted wall that served as a reference, while the other two specimens consisted of walls retrofitted with horizontal and vertical planks placed on both faces (Fig. 5a). The planks were made of select structural timber with cross-section of 180 × 40 mm and connected through the thickness of the wall using 7.94mm-diameter bolts. To maintain the original appearance of a heritage construction, installation of the planks required first making trenches so the exterior surface of timber elements could be flush with the exterior surface of the wall itself. In practice, an additional plaster would be placed on the surface of the retrofitted specimen, but such finishing touch was not implemented in this phase of the experimental program. Drilling through the thickness of the wall allowed the placement of bolts and subsequently mortar was used to fill the created orifices. Three pairs of 12.7 mm-diameter tensor rods were used to apply a total post-tensioning force of 120 kN to each retrofitted wall, thus producing an average precompression stress of 48 kN/m^2. Only one of the two retrofitted adobe specimens had the vertical timber planks and tensors anchored into the concrete foundation with the purpose of evaluating the influence of implementing such detail in the rehabilitation process. More information about construction of these adobe walls is provided in Reyes et al. [8].

5.2.2 Corner RE walls retrofitted with alternative meshes or steel plates

A total of five corner RE specimens were tested on the shake table. One specimen was unretrofitted three were each retrofitted (Fig. 5b) with alternative meshes, and the fifth specimen was retrofitted with steel plates (Fig. 5c). The three alternative meshes consisted of steel electro-welded wire mesh, high-tenacity polyester geogrid, and polypropylene geogrid. The steel wire fabric and the high tenacity polyester geogrid had tensile capacities of 90 kN/m and 306 kN/m, respectively for both parallel and perpendicular directions of the roll. The polypropylene geogrid, on the other hand, had a tensile strength of only 12.4 kN/m in the longitudinal (roll) direction and 19 kN/m in the transverse direction. To ensure adequate force transfer, each mesh was joined between the inside and outside faces of the corresponding retrofitted wall using a rectangular array of 9.5 mm-diameter through-bolts installed every 500 mm. Each bolt was connected to a 100 × 200 × 6.4 mm steel plate attached the mesh. The mesh was covered with a 30 mm-thick plaster made of lime mortar and fine sand (mixture proportion = 1:3) in correspondence to the standard practice of preserving historic features on the wall surface. Reyes et al. [11] provides specific construction procedures and details of these specimens. The plates consisted of were made of A36 steel plates 6.35 mm-thick by 101.6 mm. As shown in Fig. 5c, the steel plates were arranged on a grid on both sides of the wall and connected with 9.53 mm through-bolts every 600 mm. Detailed information on the test setup, installation of retrofitting straps, and construction procedures are provided by Reyes et al. [9].

5.3 Loading protocol and instrumentation

Dynamic testing consisted ground motions corresponding to three levels of seismic intensity with return periods of 31, 225, and 475 years according to a probabilistic seismic hazard assessment (PSHA) for the city of Bogotá, Colombia [12]. These return periods roughly correspond to levels of Damage Threshold (DT), Limited Safety (LS), and Design Earthquake (DE), respectively, according to seismic design standards. The LS and DE seismic hazards in Bogotá were found to be controlled by cortical events with moment magnitudes M_w from 6.0 to 7.6, and fault distances from 50 to 80 km. The ground acceleration series recorded at the San Francisco International Airport during the 1989 Loma Prieta earthquake (M_w = 6.9 and 59 km source-to-site distance) was selected as representative of these two seismicity levels. The original record was slightly modified using a spectrum matching technique [13] to represent the required intensities. For the DT level, on the other hand, a modified version of the 2008 Quetame earthquake ground motion (M_w = 5.7) was used [14]. Wall specimens that survived the DT, LS, and DE events were further subjected to an Ultimate Level (UL) ground motion well in excess of the 10%-in-50-year probability of exceedance event. Fig. 5 shows the acceleration series for the four ground motions recorded at the level of the foundation during each test. The measured DT, LS, and DE records are slightly different between the three alternative retrofitting schemes because these were evaluated at different phases of the project. In all cases, excitation was perpendicular to the long flange of the walls. For the corner walls retrofitted with timber planks and post-tensioning rods the UL level test was stopped after the strong phase of the ground motion due to an unforeseen problem with the instrumentation. Six accelerometers were installed on each specimen to record the acceleration response at the wall mid-height and near the top. The use of three instruments at each elevation captured the difference in response near and away the wall inner corner. An additional accelerometer was placed on the shaking table to monitor the actual acceleration history at the base of the specimen.

5.4 Selected Results

A dynamic characterization test was conducted before and after application of each the four ground motion levels. The characterization test consisted of a small pulse that induced free vibration plus a scaled version of the DT record to obtain a PGA = 0.0125g. These small imposed demands ensured a nearly elastic response of the specimen to obtain estimates of the fundamental period of vibration in the original and damaged states of the walls. The dynamic characterization was accomplished using a combined deterministic-stochastic subspace method [15].

Table 2 summarizes some relevant test results in terms of the estimated out-of-plane lateral stiffness degradation and the residual drift at the top of the wall. Results correspond to estimations and measurements made after each applied ground motion level. Values from this table allow to assess the effectiveness of a given retrofitting technique with respect to the original unretrofitted counterpart and to compare across alternative retrofitting techniques. The stiffness degradation was calculated as the square of the ratio between the measured fundamental period before any ground motion was applied and the measured fundamental period after applying the given seismic intensity level —i.e., considering the wall as an equivalent single-degree-of-freedom system. Additional test results about the observed behavior of corner walls, including damage patterns, can be found in [8, 9, 11].

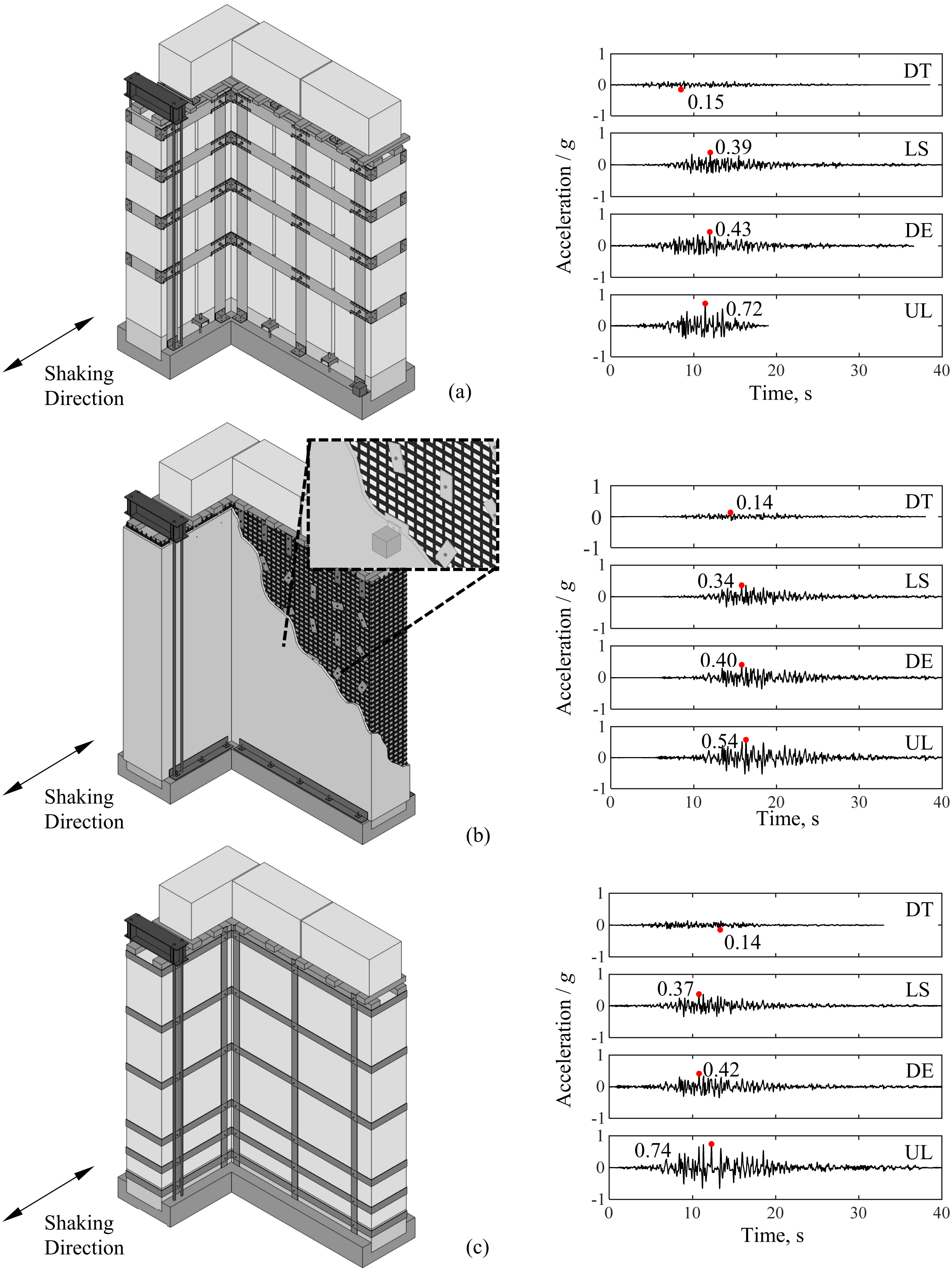


Fig. 5 – Configuration and input ground motions for shake table tests of corner walls: (a) retrofitting with timber planks and post-tensioning tensors; (b) retrofitting with meshes; (c) retrofitting with steel plates.

Table 2 – Results from shake table tests.

Material	Retrofitting scheme	Out-of-plane lateral stiffness degradation§				Residual drift (%) after ground motion			
		DT	LS	DE	LU	DT	LS	DE	LU
Adobe	None	0.38	C*	C*	C*	0.11	C*	C*	C*
	Timber planks & PT	0.59	0.49	0.38	0.29	0.00	0.23	0.23	0.64
	Timber planks & PT anchored	0.84	0.41	0.39	0.21	0.00	0.28	0.22	0.35
RE	None	0.65	0.36	C*	C*	0.27	0.28	C*	C*
	Steel mesh	0.68	0.62	0.42	0.39	0.24	0.92	1.66	3.03
	High tenacity polyester geogrid	0.78	0.40	0.39	0.31	0.27	0.42	1.32	2.17
	Polypropylene geogrid	0.95	0.49	0.42	0.38	0.29	0.51	1.73	2.46
	Steel plates	0.56	0.36	0.28	0.11	0.00	0.16	0.37	1.09

C* = wall collapsed during ground motion

§ ratio between the inferred stiffness after the ground motion and the original stiffness before any ground motion was applied

As expected, increasing levels of ground motion produced greater stiffness degradation and larger permanent drifts. One unretrofitted earthen walls collapsed when subjected to the limited safety (LS) intensity only, while the remaining specimen collapsed during the design earthquake (DE). In contrast, none of the six retrofitted walls collapsed when subjected to excitations significantly greater than the design ground motion. At low levels of seismic intensity, the unretrofitted RE wall exhibited less stiffness degradation and more residual drifts; however, they were also able to withhold higher level of shaking before collapsing as compared to the unreinforced adobe wall. It is noticed that for adobe walls retrofitted with timber plans and post-tensioning rods, anchoring the retrofitting elements to the wall foundation resulted in less degradation of the out-of-plane lateral stiffness at the LS level of seismicity. But more importantly, such detailing measure resulted in a significant reduction of the residual drift (from 0.64% to 0.35% at the ultimate level of excitation).

Initial dynamic characterization tests showed that the fundamental frequency of vibration was similar between the unretrofitted RE wall and the RE walls retrofitted with meshes [11], thus indicating that meshes did not affect the initial lateral stiffness of the walls. However, meshes had a significant influence on the lateral response of the walls after cracking occurs. The experimental results showed that at very low intensity levels (DT level), the polypropylene geogrid was the most effective with almost no stiffness degradation; however, for the LS intensity level, the steel wire mesh appeared to be more effective in preserving the out-of-plane lateral stiffness of the walls. For high ground motion intensities, the differences between specimens retrofitted with meshes were not significant. Among the three mesh retrofitting schemes, the use of steel wire fabric seems to be one of the most effective methods for delaying collapse, and thus, reducing possible injuries and life losses. It is observed that although steel plates were less efficient than meshes in preserving the out-of-plane lateral stiffness, the reduction of permanent drift make the retrofitting alternative more attractive. Timber planks, on the other hand, not only reduced the permanent drift significantly —even for a very extreme seismic event— but also provided comparable preservation of lateral stiffness as meshes. It is commonly accepted that residual drift informs the decision to demolish or not to demolish a damaged structure after an earthquake and a committee of FEMA P58 [16] proposed 1% to be the threshold. Under that criterion, all RE walls retrofitted with meshes would have to be demolished after the design earthquake. The undesirable permanent drifts were observed to occur because of the low bond capacity between the wall and the meshes, particularly at the wall corners. This limits the applicability of meshes to protect heritage structures, even though the retrofitting technique is efficient to prevent collapse.

6. Conclusions

This paper presents the results of an experimental program aimed at investigating the out-of-plane seismic response of earthen walls in heritage construction. The laboratory campaign comprised full-scale tests of fourteen specimens with thick walls subjected to either monotonic loading, cyclic loading, or dynamic loading (shake table tests). Evaluated retrofitting alternatives included the use of timber planks combined with and without vertical post-tensioning, steel and synthetic meshes, and steel plates. It was found that unretrofitted RE walls exhibit greater residual drifts at low level of shaking as compared to adobe walls, but they are also able to withstand greater ground motions. Overturning static tests revealed that use of timber planks can increase the out-of-plane of adobe walls by a factor of more than two, even in the absence of confinement from axial compression. Cyclic loading tests showed that the use of steel plates significantly increases the deformation and energy dissipation capacity of adobe walls while also delaying the drift at which cracking becomes apparent. Finally, shake table tests demonstrated that using any of the retrofitting techniques presented in this paper earthen walls can withstand ground motions well in excess that of an event with a 10% probability of exceedance in 50 years. By contrast, the unretrofitted counterparts would fail levels of seismic demand that are below the design earthquake level. Steel plates and the combination of timber planks with vertical post-tensioning were found to be most effective retrofitting techniques because they kept residual drifts small even after extreme ground motions. By contrast, meshes, while efficient to prevent collapse, exhibited residual drift that could render them unsuitable for the retrofitted of heritage construction.

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