SEISMIC PERFORMANCE OF AN INNOVATIVE TIMBER RETROFIT TECHINQUE FOR UNREINFORCED MASONRY BUILDINGS

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Abstract

An innovative timber-based retrofit solution was investigated at the EUCENTRE laboratories (Pavia, Italy) within an extensive experimental campaign on the vulnerability of existing Dutch unreinforced masonry (URM) cavity-wall terraced houses. Such structural systems are typically composed by a load-bearing single-wythe calcium silicate leaf, and by an external clay veneer with no structural function, connected to each other by steel ties. The first floor of these structures is usually built with precast reinforced concrete slabs while the second floor and roof often consist of timber joists and planks. The main objective of this retrofit technique is to enhance the seismic performance with a light, cost-effective and low-invasive intervention. The retrofit system included timber frames on which oriented-strand timber boards were nailed. Frames were mechanically connected though steel brackets and fasteners to the internal surface of masonry piers and to the building floor diaphragms. The proposed solution aimed to increase both in-plane and out-of-plane capacities of masonry piers as well as to improve the overall connections between masonry and floor diaphragms. The conceived system was firstly investigated through quasi-static in-plane shear-compression cyclic tests, performed on full-scale calcium-silicate URM piers in unstrengthened and strengthened conditions. After that, two identical full-scale two-storey buildings were dynamically tested on the shake-table at the EUCENTRE laboratories, in bare and retrofitted configurations. This paper focuses on the experimental performance of the adopted retrofit system, with emphasis on the improved seismic response of the specimens.

Keywords: Timber seismic retrofit; Unreinforced masonry; Full-scale shake-table tests; Induced seismicity; Timber diaphragm.

1. Introduction

Unreinforced masonry (URM) cavity-wall systems are a common structural solution for residential buildings in several parts of the world. This building typology is often characterized by insufficient seismic details, especially when it is adopted in regions with a low seismic hazard. Recent events have demonstrated that natural phenomena (i.e. the slip of an unknown fault [1]) and human activities (i.e. gas extraction [2, 3, 4]) can expose an entire region to ground motions with intensity higher than expected. These occurrences have moved the interest on possible retrofit solutions able to reduce the seismic vulnerability of existing buildings in these areas.

Several strengthening techniques have been proposed in literature for unreinforced masonry piers such as the application of an additional material layer to the masonry [5, 6, 7], the use of post-tensioning [8], and the application of steel elements mechanically connected to the masonry [9]. Moreover, the seismic response of URM buildings can be effectively enhanced by improving the connections between masonry and floor diaphragms [10] and between intersecting walls [11]. Overall, essential requirements that a retrofit system conceived for existing masonry buildings in low-seismicity areas should fulfil may include (i) light weight, (ii) low cost, (iii) sustainability, and (iv) reversibility. Aiming to meet these principles, a timber-based retrofit solution has been considered as a viable alternative, recognizing the timber capability to add tensile strength to the masonry and its high strength-to-density ratio. Although combined masonry-timber loadbearing structural systems have been adopted for centuries in seismic-prone regions [12], the use of timber as a retrofit solution for masonry structures have been investigated only recently, focusing on the enhancement of the in- and out-of-plane capacities of masonry piers [13, 14, 15].

Starting from these studies, an innovative timber-based retrofit system has been conceived and experimentally investigated at the EUCENTRE laboratories in Pavia, Italy. The proposed solution consists of timber frames mechanically connected to the masonry surface, coupled with oriented-strand boards (OSB). The retrofit solution was first investigated through quasi-static in-plane cyclic tests on a full-scale masonry pier [16], and then was applied to a full-scale two-storey building subjected to dynamic shake-table tests [17, 18, 19]. The same prototypes were tested also in unstrengthened conditions to fully appreciate the effectiveness of the retrofit system on both an individual structural component and a whole building system. This paper describes the concepts at the base of the proposed retrofit system, and compares the experimental results obtained from the performed tests, emphasizing on the improved seismic response of the specimens. All processed data and instrumentation schemes are available at http://www.eucentre.it/nam-project.

2. Retrofit system features

The adopted retrofit system was conceived starting from the out-of-plane strengthening solution proposed by Dizhur et al. [14] and by Giarretton et al. [15] based on the use of vertical timber strong-backs. Such retrofit solution has been further extended to improve also the in-plane force and displacement capacities of piers, as well as to overcome the lack of connections between masonry and floor diaphragm systems, recognized as one of the most relevant weaknesses showed by existing URM buildings designed with insufficient seismic details [19, 20].

The proposed strengthening system consists of a timber frame mechanically connected to the masonry inner surface on which OSB are nailed. The frame includes timber vertical posts, horizontal nogging elements, and top and bottom sill plates which complete the frame allowing its connection to the floor and foundation systems (Fig. 1). The effectiveness of this system is strongly affected by the quality of connections between timber frames, masonry elements and floor systems.

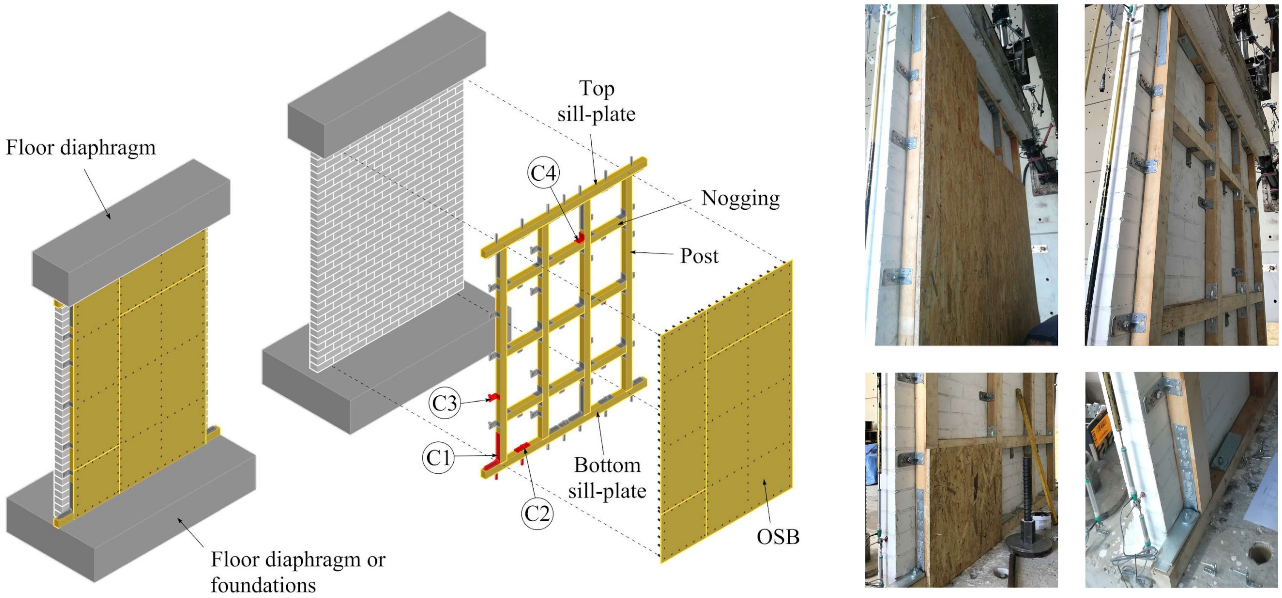


Fig. 1 – Retrofit system components and layout

As illustrated in Fig. 1 four different connection types were employed: C1 indicates the tie-down anchors that link together posts, sill plates, and floors (or foundations); C2 refers to further connections between sill plates and floor or foundation systems, added to prevent sliding-shear mechanism; C3 indicates the connection between timber frames and masonry walls, while C4 identifies the connection between timber frame members.

The masonry wall out-of-plane capacity is improved by the vertical posts which, fastened to the timber sill plates, behave in flexure as strong-backs transferring the inertia forces from the panel to the adjacent floor systems (i.e. diaphragm or foundations). On the other hand, the pier in-plane response is enhanced by both timber frames and OSB: the retrofit system and the masonry, forced by connections C3 to experience the same deformations, respond to the lateral loads behaving as springs in parallel. In particular, vertical posts and their tie-down connections (C1) provide the flexural contribution to the retrofit system in-plane strength, while nailed OSB contribute to the timber system in-plane shear strength.

3. In-plane quasi-static tests

3.1 Overview of the tested specimens

The first investigation on the effects of the newly proposed retrofit system was carried out at the EUCENTRE laboratories [16] on two identical full-scale calcium-silicate (CS) piers, one in bare and one in retrofitted configuration. The specimens could represent the first-storey longest resisting pier of a cavity-wall terraced house typical of the North-East part of The Netherlands. They were tested up to their ultimate conditions through quasi-static shear-compression cyclic tests, under the same vertical overburden stress of 0.5 MPa and double-fixed boundary conditions, aiming to quantify the influence of the timber system on the pier displacement and strength capacities.

The two masonry piers were 2.70-m high, 2.00-m long and 0.10-m thick, and consisted in 33 courses of single-wythe CS bricks with average dimensions of 210x100x70 mm, and 10-mm-thick mortar joints (Fig. 2). They were built simultaneously on reinforced concrete (RC) footings and matured under the same environmental condition. Then one of the two piers was retrofitted with the proposed timber system. It consisted of 80x60 mm timber elements oriented with the smallest dimension perpendicular to the masonry surface. 18-mm-thick OSB layer was nailed to the frame with 4-mm-diameter, 75-mm-long anker nails [21] at 100-mm-spacing (Fig. 2b), according to the American specifications for timber shear-walls [22]. The member sizes were defined to reach an acceptable compromise between the retrofit structural efficiency and its invasiveness considering the application to real buildings (see section 4). Connections C1, C2, C3 and C4 were realized with steel brackets [23] fastened to the timber elements by means of 5-mm-diameter, 70-mm-long screws [21] and to the masonry or RC elements by class 8.8 threaded rods with diameter of 10 mm and 12 mm, respectively.

3.2 Material mechanical properties

Masonry material properties were determined through mechanical characterization tests performed at DICAr laboratory of the University of Pavia on assemblies that reached at least 28 days of maturation.

Calcium silicate bricks were tested in compression and bending according to EN 772-1 [24], to obtain their compressive (f_b) and tensile (f_{bt}) strengths of 19.8 MPa and 2.5 MPa. The tensile (f_t) and compressive (f_c) strengths of mortar were determined respectively as 1.74 MPa and 5.06 MPa, according to EN 1015-11 [25]. A masonry compressive strength (f_m) equal to 10.1 MPa and a secant elastic Young modulus at 33% of its compressive strength (E_m) of 6593 MPa were obtained by testing masonry wallettes in compression according to EN 1052-1 prescriptions [26]. The initial shear strength (f_{v0}) and the friction coefficient (μ) were obtained according to the recommendations of EN 1052-3 [27], and were equal to 0.62 MPa and 0.71 respectively. The masonry density (ρ) was equal to 1850 kg/m^3.

The employed timber was red solid fir (*Picea abies*) of class S10/C24 according to EN 14081-1 [28] with a density of 517 kg/m^3. The specified characteristic compressive strength parallel to fibers ($f_{c,0}$) was 21 MPa, the characteristic tensile strength parallel to fibers ($f_{t,0}$) was 14 MPa, and the mean Young modulus ($E_{0,mean}$) was 11000 MPa. The OSB were classified as OSB/3 according to EN 300 [29], with a density of 572 kg/m^3. The steel angles used for tie-down connections C1 had characteristic tensile strength of 11.6 kN [23].

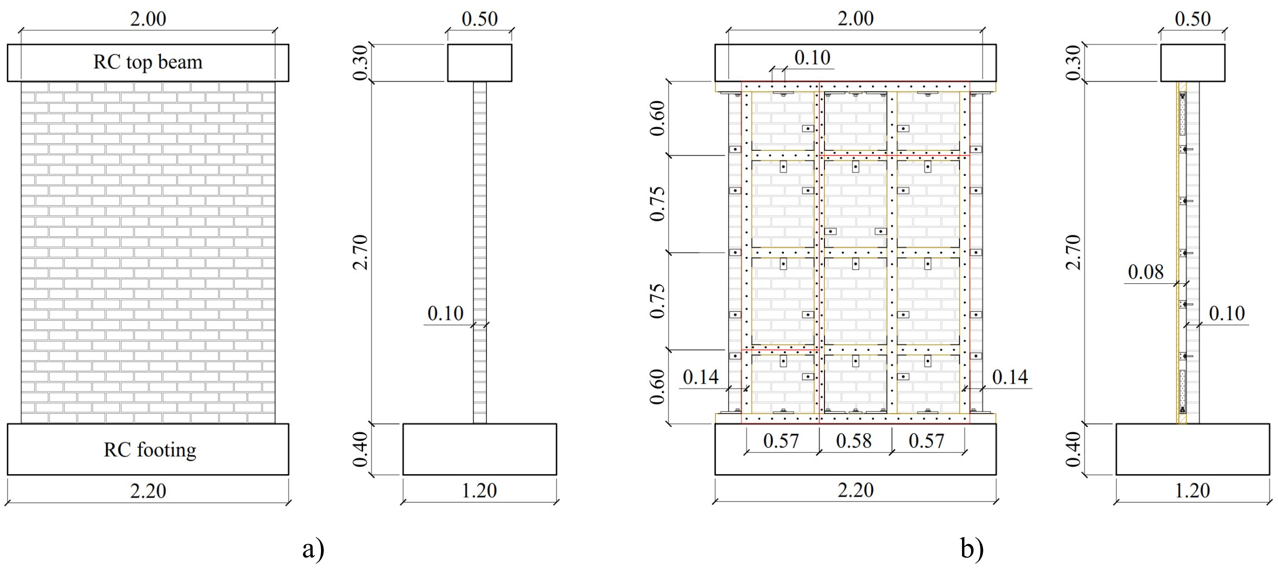


Fig. 2 – Masonry specimens: a) bare condition; b) retrofitted configuration.

3.3 Test results and retrofit benefits

The bare masonry pier lateral response was characterized by an initial rocking behaviour, followed by a shear-sliding one. The pier showed the first horizontal cracks at drift ratio (θ) of 0.075% and reached a maximum base shear of 78 kN at a drift ratio of 0.2%. After that, the development of a full-length crack at the top courses of the pier prevented the maintaining of the flexural behaviour. The test was stopped at a drift ratio of 0.75% when the pier was no longer able to withstand vertical loads. Fig. 3a shows the experimental damage observed at ultimate conditions.

The retrofitted pier responded to horizontal loads predominantly in flexure for the whole duration of the test. The first cracking occurred at a drift ratio of 0.075%, as observed for the bare pier. Then, from a drift ratio of 0.15%, the toe-crushing mechanism started to develop and became more and more significant until the end of the test. The first damage to a retrofit component was noticed after the cycles performed at a drift ratio of 0.60%: the steel angles of tie-down connectors (C1) at the top of the specimen buckled in compression. The maximum shear strength of 110 kN was recorded at a corresponding drift ratio of 0.80% (Fig. 3b). At a drift ratio of 1.50% the shear deformation reached by the pier could not be accommodated by the masonry and caused the formation of a diagonal shear crack. The test continued up to an ultimate drift of 2.00% (Fig. 3c), after which the masonry was no longer able to sustain vertical loads, which were widely supported by the timber system.

At ultimate conditions (Fig. 3a, Fig. 3c), both piers presented significant toe crushing, more pronounced on the retrofitted one. However, for the bare pier the mechanism was interrupted due to the combination of its hybrid (rocking/sliding) behaviour and induced out-of-plane instability. Instead, for the retrofitted pier, the mechanism was pushed further exploiting the full flexural rocking capacity of the specimen thanks to the timber system, which concentrated the whole damage in tie-down elements (C1). Moreover, although the retrofitted pier was heavily damaged, the timber system helped the masonry to maintain gravity load resistance, while the bare pier lost its capacity of bearing vertical loads. The experimental backbone curves are reported in Fig. 3d: the retrofit system allowed the strengthened specimen to sustain an ultimate displacement and a lateral strength, respectively 167% and 35% higher than the ones exhibited by the bare pier. Further details on the pier experimental responses can be found in [16].

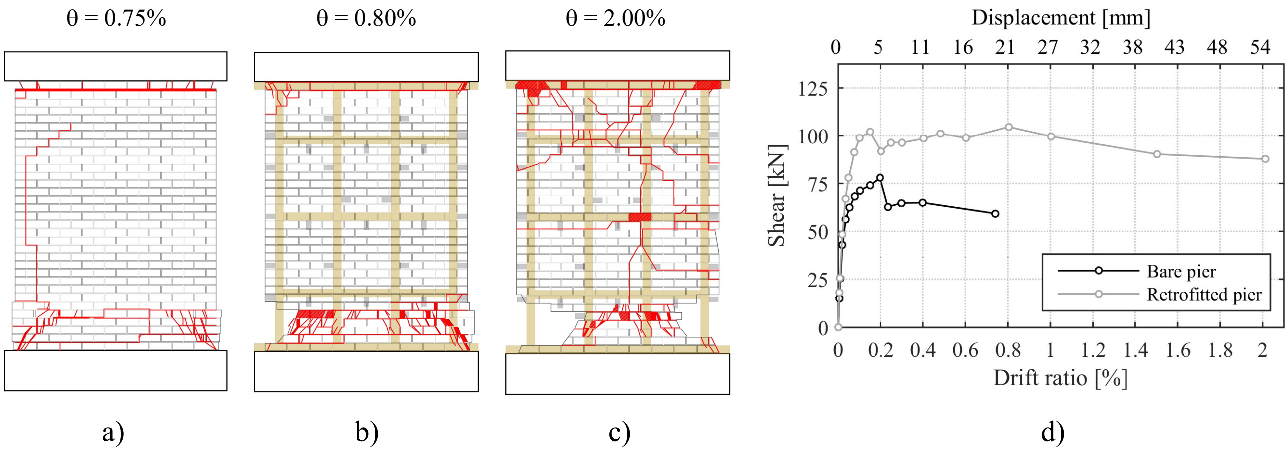


Fig. 3 – Experimental crack patterns: a) bare pier at ultimate conditions; b) retrofitted pier after the cycles corresponding to the ultimate condition of the bare pier; c) retrofitted pier at ultimate conditions; d) bare and retrofitted pier backbone curves.

4. Full-scale shake table tests

Following the investigation of the retrofit system effects on the displacement and strength capacities of an isolated pier, the intervention was applied to a full-scale masonry building. The prototype was a replica of a typical two-storey URM Dutch terraced-house end unit, and was tested incrementally on the shake-table in bare conditions in June 2018 at EUCENTRE [17]. Then, another identical structure was built by the same masons, retrofitted with the proposed timber system, and tested in December 2018 using the same incremental dynamic input [18]. The comparison of the experimental results allowed an effective assessment of the influence of the retrofit system on the seismic capacity of the building.

4.1 Specimens overview and retrofit system details

The prototype was 7.8-m high and presented plan dimensions of 5.9 x 5.6 m, with the longest and most vulnerable direction oriented parallel to the shaking direction (Fig. 4a). Front (West) and back (East) longitudinal façades were characterized by a large amount of openings asymmetrically distributed (Fig. 4a, Fig. 4b). The South transversal wall had no openings, simulating a party wall, while the North façade presented a trapezoidal window at the top of the gable wall (Fig. 4b). The structural system relied on URM cavity-walls consisting in an internal single-wythe, 100-mm-thick loadbearing CS leaf and an external single-wythe, 100-mm-thick clay veneer, both with 10-mm-thick mortar joints. 3.1-mm-diameter steel ties provided the connection between the two masonry leaves with a density of 1 tie/m^2.

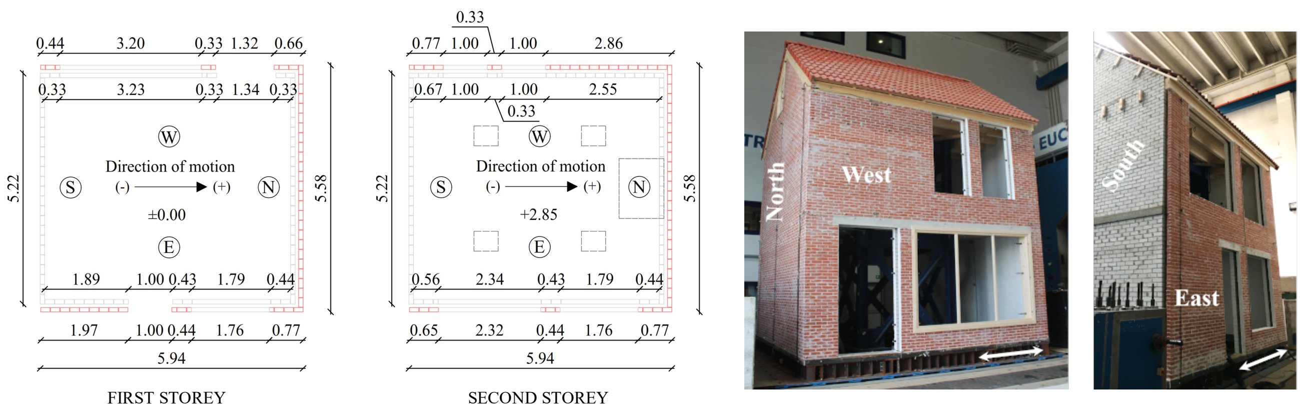
The specimen was built on a composite concrete/steel foundation firmly bolted to the shake-table. The first floor was characterized by a rigid 160-mm-thick RC slab while the second one by a flexible timber diaphragm. Both the diaphragms were supported only by the North and South CS transverse walls and presented a stair-case opening and four holes to accommodate a reference steel frame (Fig. 4a). The structure was completed by a 39° pitched timber roof. The connections between second-floor and roof 100x240-mm timber joists with the supporting walls were realized through L-shape steel anchors, with diameter of 14 mm. 185x18-mm tongue-and-groove planks were nailed to the joists by pairs of 2-mm-diameter, 60-mm-long nails.

The retrofit system was applied to the full-scale building keeping the same timber element sizes, OSB thickness, nail spacing and connectors employed for the retrofitted pier tested in-plane as described in section 3.1 (Fig. 5a, Fig. 5b). Aiming to keep the retrofit system as light as possible, different retrofit strategies were adopted on longitudinal and transversal sides. In particular, nogging elements and OSB were not applied to transversal solid walls due to their significant in-plane capacity (Fig. 5c). An 18-mm-thick OSB layer was nailed to the second-floor flexible timber diaphragm, fastened to existing joists and additional timber blocking beams by 4-mm-diameter, 75-mm-long anker nails [21], according to the American provisions for timber diaphragms [22]. The roof diaphragm was initially not stiffened, but loose safety steel cables were subsequently tied to form cross-braces between the ridge beam and the second floor, when the gable out-of-plane displacements became excessive.

Considering the application of the proposed retrofit system to a full-scale URM building, it is important to notice that the improvement of connections between masonry elements and floor diaphragms plays a fundamental role in the enhancement of the building seismic performance. Indeed, by triggering a global box-type behaviour it is possible to prevent the building to develop undesired local mechanisms which can dramatically affect its seismic capacity. This aspect required further masonry-to-floor systems connection detailing and implied an additional assessment of the floor diaphragm strength capacities.

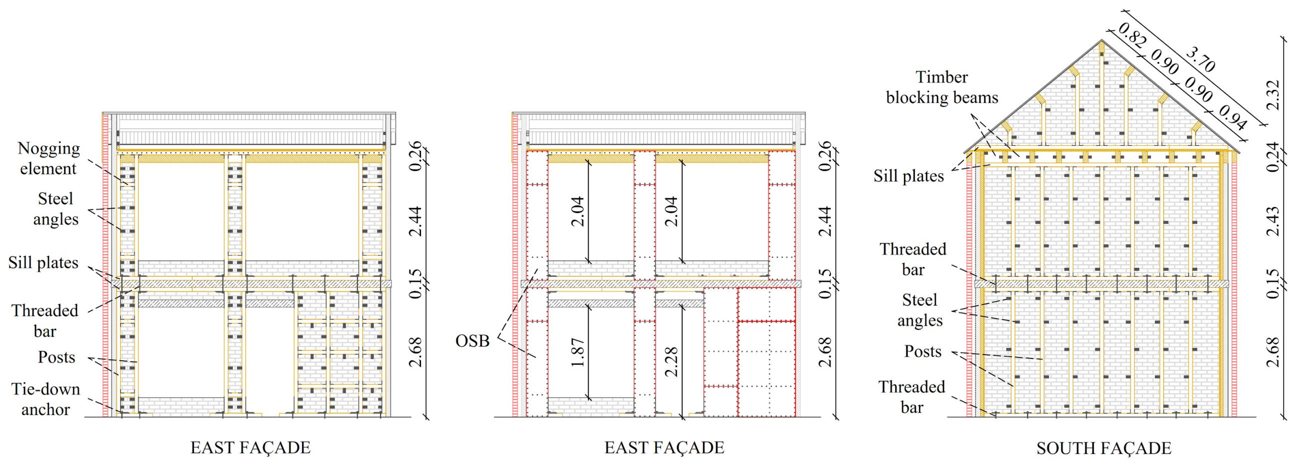
For connections C1 and C2 with the first-floor diaphragm, threaded bars crossing the RC slab through core-holes were bolted to the first-storey top sill plate and to the second-storey bottom sill plate (Fig. 6a). Along the transverse walls, vertical posts were connected to sill plates through simple C4 timber-to-timber connections, while C2 connections tied the sill plates to the RC slab or to the foundations (Fig. 6b). Concerning the second-floor diaphragm, the second-storey top sill plates were screwed densely to the inner-leaf spreader beams along the longitudinal walls (Fig. 6c). Timber blocking beams inserted between floor-joists along the transverse walls allowed connecting the second-storey top sill-plate and the gable bottom sill plate to the diaphragm (Fig. 6d). The vertical posts attached to the gables were connected directly to the roof purlins, since no top sill-plate was provided there (Fig. 5c).

Moreover, the density of steel ties was increased up to 5 tie/m^2 to simulate the enhancement of the connection between inner- and outer-leaves. For the same reason, the corresponding spreader beams were fastened together by pairs of diagonally crossing, 8-mm-diameter, 360-mm-long timber screws at 500-mm spacing.



a)
b)

Fig. 4 – Bare building overview: a) plan views; b) overviews of the building prototype (white arrows indicate the shaking direction). Units of m.



a)
b)
c)

Fig. 5 – Retrofitted specimen elevations: a) timber frames applied to longitudinal façades; b) OSB nailed to frames on longitudinal façades; c) timber frames applied to transverse façades. Units of m.

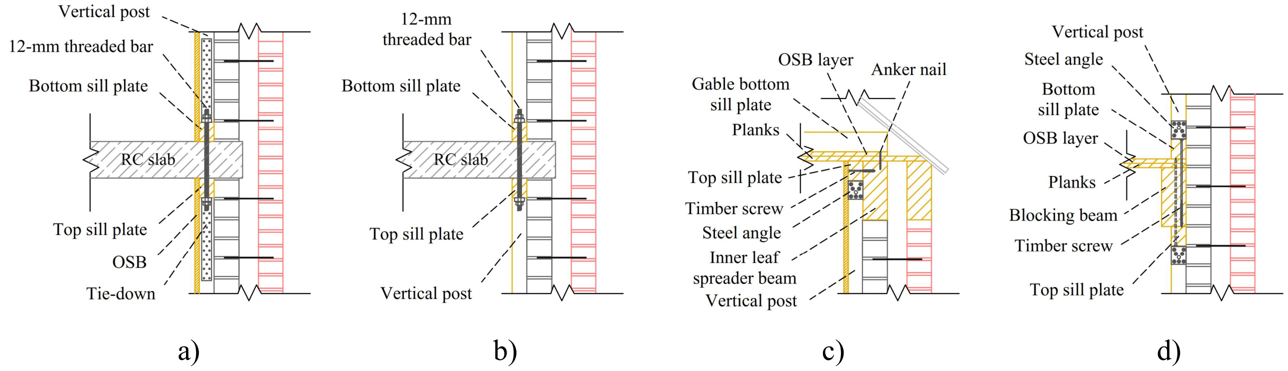


Fig. 6 – Connection details: a) longitudinal walls-to-first floor; b) transverse walls-to-first floor; c) longitudinal walls-to-second floor; d) transverse walls-to-second floor.

4.2 Mechanical properties and masses

All masonry mechanical properties were determined through characterization tests performed at DICAr laboratory of the University of Pavia. The bare building masonry came from the same batch of the piers tested in-plane, therefore material properties can be referred to section 3.2. It is worth to note that only mechanical properties related to the calcium-silicate inner leaf are reported since the clay veneer had no intended structural function.

Concerning the retrofitted prototype masonry, the tensile (f_t) and compressive (f_c) strengths of mortar were determined respectively as 1.39 MPa and 3.97 MPa, according to EN 1015-11 [25]. A masonry compressive strength (f_m) equal to 10.05 MPa and a secant elastic Young modulus at 33% of its compressive strength (E_m) of 7319 MPa were obtained according to EN 1052-1 prescriptions [26]. The initial shear strength (f_{v0}) and the friction coefficient (μ) were obtained according to the recommendations of EN 1052-3 [27], and were equal to 0.49 MPa and 0.39 respectively. The retrofit system consisted of the same red solid fir timber elements of class S10/C24, OSB belonging to category OSB/3, and steel connectors employed for the in-plane tests. Related mechanical properties can be found in section 3.2.

Calcium silicate and clay masonry had average densities respectively of 1850 kg/m^3 and 2020 kg/m^3. The mass of the inner masonry leaf was 17.7 t while the one of the outer leaf was 13.9 t. First and second floors had masses of 11.2 t and 1.9 t, respectively, including additional weights. The complete roof had mass of 2.8 t. The total weight of the bare building was 47.5 t while the one of the retrofitted specimen was 49.1 t, where the difference of 1.6 t is due to the presence of the installed timber system.

4.3 Testing protocol

The building specimens were subjected to an incremental dynamic test, applying to the shaking table a series of motions of increasing intensity to assess damage evolution, failure modes, and ultimate capacity of the buildings. The input signal consisted of an acceleration time history representing a realistic ground motion of the induced seismicity of the North-East part of The Netherlands. A single-component earthquake accelerogram, termed EQ-NPR, with PGA = 0.31 g and short significant duration $D_{s,5\text{-}75}$ = 1.82 s [19], was selected upon spectrum-compatibility with the uniform hazard spectrum (UHS) at 2475-years return period for the site of Loppersum (lat. +53.33, long. +6.75; [30]). The acceleration amplitude of the original input signal was progressively scaled to 33%, 50%, 66%, 85%, 100%, 133%, 166%, 200%, and 266%.

4.4 Test results and retrofit benefits

The bare specimen exhibited a seismic behaviour strongly influenced by local mechanisms. Specifically, the insufficient connections between the flexible second-floor diaphragm and the masonry caused the onset of sliding between the two systems, preventing the flow of inertia forces from the top to the bottom of the building and concentrating the displacement demand, and the consequent damage, at the second storey. The test was terminated after testing with EQ-NPR scaled at 133% (*PGA* = 0.39 g) since the structure presented significant residual displacements and was deemed to be very close to lose its static stability (Fig. 7a).

Instead, the retrofitted specimen exhibited a global box-type response throughout the whole incremental test, thanks to the improved connection between masonry and floor systems and to the increased in-plane and out-of-plane strength and displacement capacities of masonry piers. Up to run EQ-NPR-100% (*PGA* = 0.31 g) the retrofitted specimen accumulated damage only in the roof-gable subsystem, which showed higher flexibility compared to the first and second storey [19, 31]. After test EQ-NPR-133% (*PGA* = 0.39 g), the roof-gable subsystem was stiffened by steel cables, to limit gable out-of-plane displacements. More importantly, at this stage diagonal stair-stepped cracks were observed on the first-storey transverse walls, denoting the onset of a global torsional response (Fig. 7b). It is worth to highlight that the bare specimen reached near-collapse conditions under the same shaking intensity.

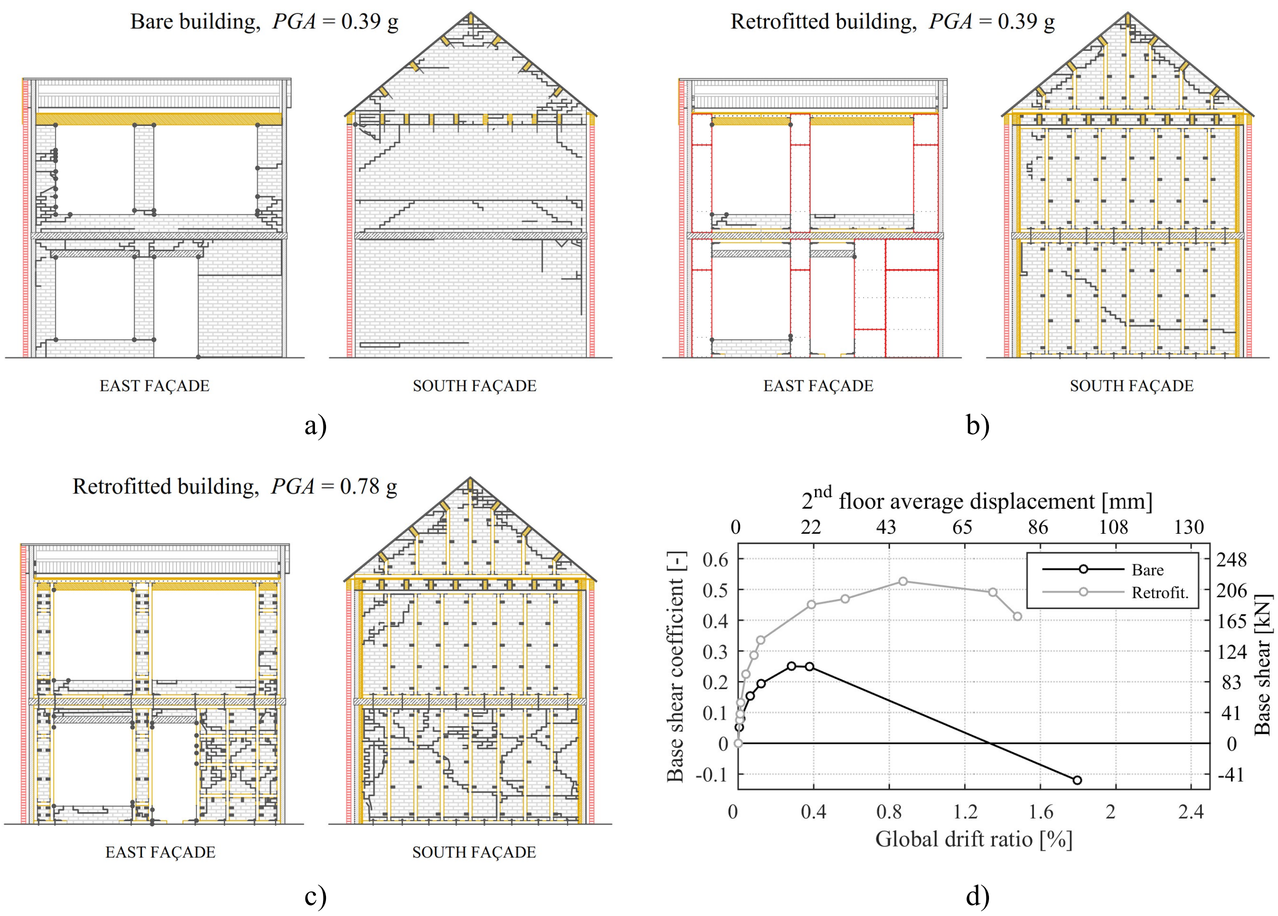


Fig. 7 – Damage patterns: a) bare building at ultimate conditions; b) retrofitted building after shaking with the last run performed for the bare specimen; c) retrofitted building at ultimate conditions; d) bare and retrofitted specimen backbone curves.

The following increments of ground motion amplitude accentuated the first-storey damage due to torsional effects, denoting a weak-storey mechanism, and caused a significant accumulation of residual displacements. The test was stopped after the run performed under EQ-NPR-266% (PGA = 0.78 g), as the specimen was deemed very close to near-collapse conditions. Fig. 7c shows the damage pattern at ultimate conditions. The damage was concentrated at the first-storey, with significant residual displacements, as opposed to the second one, which showed only diffused hairline cracks. The strengthening intervention allowed the retrofitted specimen to sustain twice the scaling of the ground motion applied to the bare one: the full capacity of resisting piers was fully exploited thanks to the improvement of masonry-to-floor systems connections.

Further considerations can be made through the experimental backbone curves overlap (**Fig. 7d**). The base shear was computed neglecting the non-oscillatory mass of the bottom half of the first-storey walls, while the base shear coefficient was computed dividing the shear force by the corresponding weight. The global drift ratio was computed dividing the second-floor average displacement by its height above the foundation. The points corresponding to the maximum base shear of each run in the negative direction were considered, as well as the one associated with the maximum displacement of the final test.

As expected, the initial stiffness of the specimens was not affected by the timber system, due to its high flexibility compared to the masonry. In terms of strength capacity, the intervention allowed the building to sustain more than two times the base shear coefficient sustained by the bare building, recorded respectively as 0.53 and 0.25. Focusing on the displacement capacities, despite the bare building apparently showed a higher experimental ultimate drift, it is worth to notice that it was reached with an inverted sign of the base shear, denoting the loss of static stability recovered only by the inversion of the ground motion sign. On the contrary, the retrofitted building ultimate drift was reached with a base shear close to the maximum recorded one. Further details on the specimen seismic responses can be found in [17, 18, 19].

5. Conclusions

This paper discussed an innovative light timber solution for the seismic retrofit of URM buildings to improve both in-plane and out-plane pier capacities, and masonry-to-diaphragm connections. It consists in timber frames mechanically connected to the masonry, to the foundations, and to the floor systems, with an OSB layer nailed to them. The effectiveness of the system was investigated firstly by testing in-plane two identical isolated full-scale masonry piers, one in bare and one in retrofitted configurations, then by applying it to a replica of a full-scale two-storey building previously tested in bare conditions on the shake-table.

The masonry pier specimens were subjected to in-plane quasi-static cyclic tests under the same overburden stress and boundary conditions. The bare pier exhibited a hybrid behaviour, initially characterized by a flexural response followed by a shear-sliding one with decreased strength, while the retrofitted pier was forced by the timber system to maintain a predominant flexural behaviour exhibiting an appreciable increase of strength and displacement capacities of 33% and 167%, respectively.

The application of the system to the full-scale building required additional detailing of the masonry-to-floor systems connections to achieve a global response, preventing the occurrence of local mechanisms. Indeed, the bare building experimental response showed an independent behaviour of the second-floor system that slid on the second-storey piers without engaging their resistances, evidencing a severe lack of connections. As a consequence, the displacement demand and damage concentrated at that storey, leaving the first one barely damaged. Instead, the retrofitted building showed a global box-type response up to the end of the incremental dynamic test, concentrating the damage at the first storey, denoting a weak-storey mechanism. The combined improvement of connections and pier capacities allowed the retrofitted building to develop a torsional response that fully engaged the strength of all structural elements, including the in-plane strength of transverse solid walls. The displacement demand was homogeneously distributed throughout the building height. As final result, the retrofitted specimen sustained twice the scaling of ground motion applied to the bare specimen, and did not experience static instability as shown by the unstrengthened prototype. All processed data and instrumentation schemes can be requested at http://www.eucentre.it/nam-project.

6. Acknowledgements

This work is part of the EUCENTRE project "Study of the vulnerability of masonry buildings in Groningen", within the research programme framework on hazard and risk of induced seismicity in the Groningen province, sponsored by the Nederlandse Aardolie Maatschappij BV (NAM). The data post-processing was also partially funded by the DPC-ReLUIS within the framework of the Work-Package-5 2019-2021: "Interventi di rapida esecuzione a basso impatto ed integrati". The authors would like to thank all parties involved in this project: the DICAr Laboratory of the University of Pavia and the EUCENTRE Laboratory, which performed the tests, the DPC-ReLUIS, and the partner NAM. The valuable advice of R. Pinho, G. Magenes and A. Penna was essential to the project and is gratefully acknowledged.

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