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FULL SCALE SHAKING TABLE TEST ON A URM CAVITY WALL TERRACED HOUSE BUILDING

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Abstract

Keywords: Shaking table test, Full-scale building, URM cavity walls, Calcium Silicate bricks.

1. Introduction

The results presented in this manuscript are part of a wider research project aimed at assessing the vulnerability of buildings typical of the Groningen region (located in Northeast Netherlands). This area, historically not prone to natural seismic events, in the last two decades was subjected to earthquake ground motions induced by reservoir depletion due to gas extraction. The most relevant event was a local Magnitude 3.6 earthquake that occurred on 16 August 2012 near Huizinge, above the central part of the Groningen field [1]. Buildings not specifically designed for seismic actions have been exposed to earthquake phenomena. Unreinforced masonry (URM) buildings represent the large majority of the local existing building stock (almost 90%).

Currently, very limited data is available on the seismic response of construction typologies specific to Dutch practice. An experimental campaign, initiated in 2015, investigated the performance of structural components, assemblies and systems typical of building typologies present in the Groningen area. The experimental campaign included *in situ* mechanical characterization tests [2] and laboratory tests as characterization tests on bricks, mortar and small masonry assemblies; in-plane cyclic shear-compression [3] and dynamic out-of-plane tests on full-scale masonry piers [4]. Two full-scale shaking table tests have been conducted in 2015 and 2016 on two different URM typologies on the shaking table test facility of the EUCENTRE laboratory, in Pavia, Italy [5, 6]. In particular, this paper presents a study on the seismic behaviour of a cavity-wall terraced house specimen. With the aim of reproducing the behaviour of this type of existing masonry building, an incremental dynamic test was carried out up to the near collapse limit state of the specimen. This architectural typology represents more than half of the URM building stock of the region under consideration. They are usually two-storey buildings with openings on only two of their sides, consisting of several structurally independent side-by-side units (4 to 6). The majority of this architectural typology is built with cavity walls, as this construction system became a sort of standard after World War II.

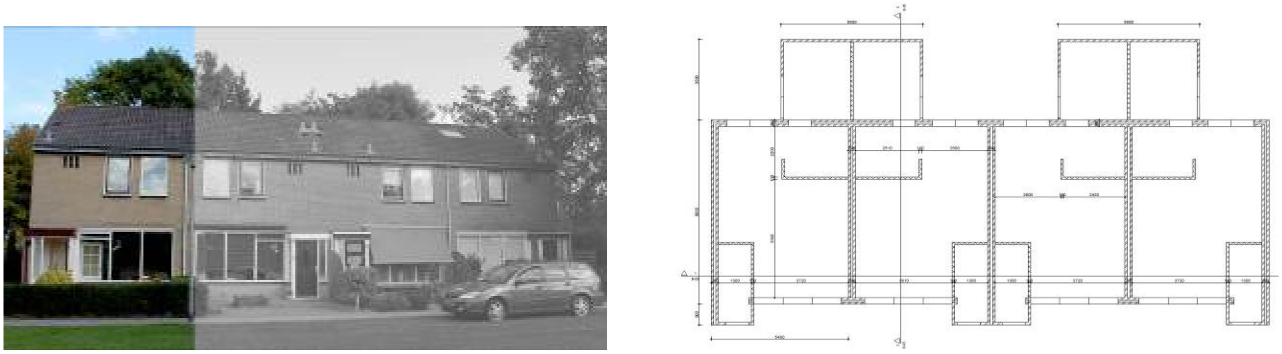
A cavity-wall building is a form of construction where a cavity is left between the two leaves of bricks. Sometimes insulating material is inserted in the cavity. The external leaf of a cavity wall is often a brick veneer wall without any load bearing function, while the internal leaf has a load-bearing function, carrying the vertical loads transmitted by the floors and roof. It is common for the inner leaf to be constructed with different materials than the outer leaf. In several European countries an example of this solution is to have the inner wall made of calcium silicate units/bricks, while the outer wall uses clay bricks. Leaves on either side of a cavity wall are typically connected by regularly spaced metal cavity ties, which can vary in material, shape and spacing. Because of their relatively lightweight, good thermal insulation properties and effective protection against driving rain, cavity walls are widely used in Central and Northern Europe countries, especially for residential construction. Information on the seismic behaviour of cavity walls is quite limited, and mostly related to earthquakes occurring in Australia (Newcastle, 1989) and New Zealand (Christchurch sequence, 2010-2011) [7]. Furthermore, a shaking table test on a cavity-wall specimen with load-bearing concrete blocks was performed by Degée *et al.* [8].

This manuscript describes the geometric and mechanical characteristics of the specimen (Section 2), the testing protocol and the applied shaking table motions (Section 3) and the test results in terms of damage evolution and hysteretic responses (Section 4).

2. Specimen Characteristics

The building specimen was intended to represent the end unit of a URM cavity walls terraced house system of the late 70s, without any particular seismic detailing. This residential typology is characterised by wide openings in the front and back sides that make them vulnerable to seismic action in the longitudinal direction; the transversal walls, that divide units, are double-leaf walls without any openings, capable of resisting significant in-plane seismic actions. For this reason, the shaking table test was carried out applying an acceleration time history along the longitudinal direction. Fig. 1 shows a picture of a classical detached house and its plan view.

Adjacent units are structurally detached, and the discontinuous slabs rest only on the load-bearing walls of the individual units. Each unit is therefore completely self-supported by transverse walls and structurally independent from the other units. The only common walls are the outer veneer walls. For this reason, it was possible to test on the shaking table a representative sub-volume (one end unit) of an entire terraced house (as shown in the coloured part of Fig. 1a).



(a) (b)

Fig. 1 – A typical terraced house in Loppersum, Groningen, NL:(a) picture of the main façade; (b) plan view

2.1 Geometry of the specimen

The test-house is a full scale two-storey building, with a timber roof and reinforced concrete (RC) slabs. The specimen was built directly on the shake-table of the EUCENTRE Lab (shown in Fig 2a). It is 5.82 m long, 5.46 m wide and 7.76 m high with a total mass of 56 t. The walls, supported by a mixed steel-RC foundation, consist of two unreinforced masonry leaves: the inner leaf, made of Calcium Silicate (CS) bricks, has a load-bearing function supporting the two RC floors. First and second-floor slabs (10.3 t and 11.0 t respectively) span between the two transverse (North and South) inner CS walls. The external leaf, instead, is a clay brick veneer without any load bearing function. The inner CS wall runs around the entire perimeter, while the outer clay brick leaf is not present in the South façade. Fig. 2b and Fig. 2c show the specimen ground and first floor plan views.

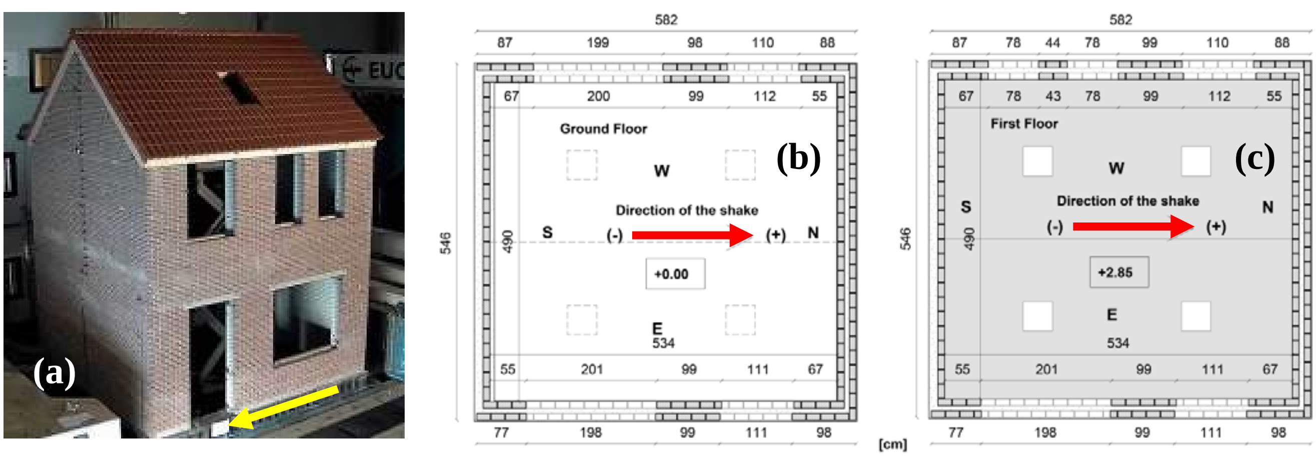


Fig. 2 – (a) Full-scale test-building (NW view); (b) plan view of the ground; (c) plan view of the 1st floor. Arrows indicate the assumed positive direction of shaking

An air gap of 80 mm is left between the two walls for insulation purposes, as in common practice. L-shape steel ties with a diameter of 3.1 mm and a length of 200 mm were inserted in 1 cm thick mortar bed-joints during the laying of the bricks in order to connect the two masonry leaves. Fig. 4a and Fig. 4d show respectively the L steel ties and a detail of the connection between the two walls. Two gable walls in the transverse façades (North and South) supported a 43° pitched timber roof. At the first floor, reinforced pre-cast concrete lintels were placed above openings for doors and windows on both inner and outer walls. The dimensions of the lintels were 160x100 mm for CS walls and 110x100 mm for clay walls. The length of lintels is 133 cm for the smaller openings and 222 cm for the wider ones.

Fig. 3 shows the elevation views of the specimen CS inner wall façades. The blue dots indicate the location of the steel ties connecting the two walls.

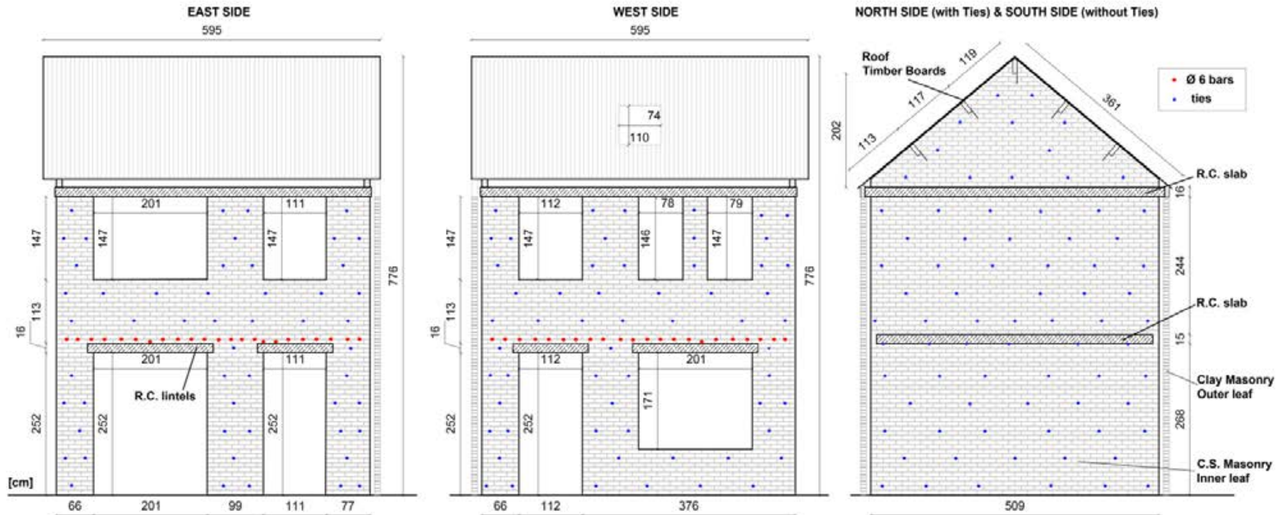


Fig. 3 – Elevation views of the test-house – CS masonry inner leaf

A rigid steel-frame safety structure installed inside the test-house acted as a safety support system for the two slabs in case of partial or global collapse of the test specimen and also provided a rigid reference system for a direct measure of the specimen displacements. The frame did not have any connection with the building since its columns passed through the two slabs by means of 45 cm square holes, large enough to accommodate significant deformations of the testing specimen.

2.2 Building construction details

It is well known among the engineering community that construction details can significantly affect the seismic response of a structure, especially URM buildings. Observation of damage caused by major earthquakes as well as laboratory tests [9] have shown that the role of connections between horizontal and vertical structural elements is of primary importance for ensuring a good structural performance. The construction details of the specimen were built to be representative of the Dutch common construction practice in the 60s and 70s. Fig. 4 presents some pictures of the specimen construction.

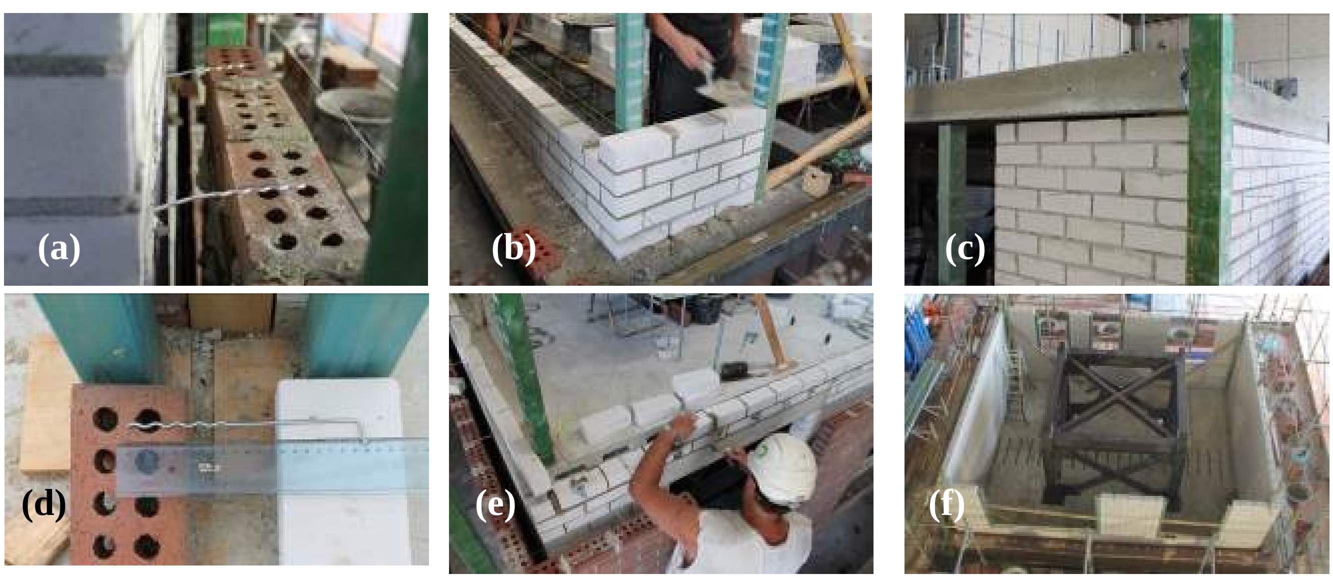


Fig. 4 – Cavity wall construction: (a) connection detail; (b) building phase of CS inner leaf; (c) slab of 2nd floor; (d) steel tie; (e) inner wall above the 1st floor; (f) steel-frame inside the specimen

The connection between the first-floor slab and the inner CS longitudinal walls (East and West) is ensured by means of threaded bars, 6 mm in diameter (Fig. 5a and Fig. 5b). Such connectors are also indicated in Fig. 3 by red dots. There was no direct connection between the outer veneer wall and the first-floor slab. The second-floor slab was not directly supported by the longitudinal walls; the gap between the slab and the inner CS longitudinal walls was filled with mortar after the removal of the temporary support and the attainment of the slab deflection. Similarly, the external timber roof beams were not in contact with the longitudinal clay walls, but they were attached to the second-floor slab edge by means of 100 cm spaced ϕ 10 threaded bars. Also in this case the gap between the timber beam and the outer clay wall was filled after the attainment of the slab deflection (Fig. 5c and Fig. 5d). Such details were adopted in order to simulate a configuration very common in the building stock. This solution resulted in almost no vertical load being transmitted to the longitudinal walls under static conditions.

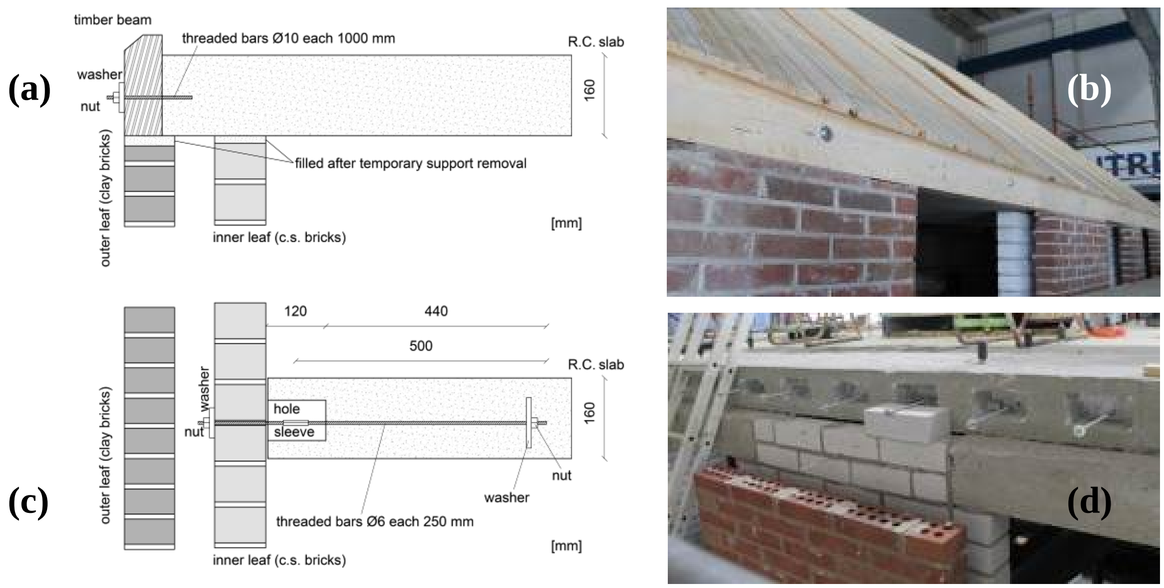


Fig. 5 – Connection between the 2nd floor slab and cavity longitudinal walls (a, b) and between the 1st floor slab and CS longitudinal walls (c, d)

The timber roof is a simple structure consisting of beams and cross-boards nailed to the main elements. It was composed by one ridge beam, two wall plate beams on top of the longitudinal outer leaves and two girders per side in between the ridge and the wall plate beams at approximately every 1.2 m. 1.8 cm thick and 18.2 cm wide planks were nailed on top (two 60x2 mm nails at each intersection). The timber beams of the roof were supported by the transverse inner CS leaves (North and South walls), and this connection was further improved by the presence of steel anchors, as shown in Fig 6c.

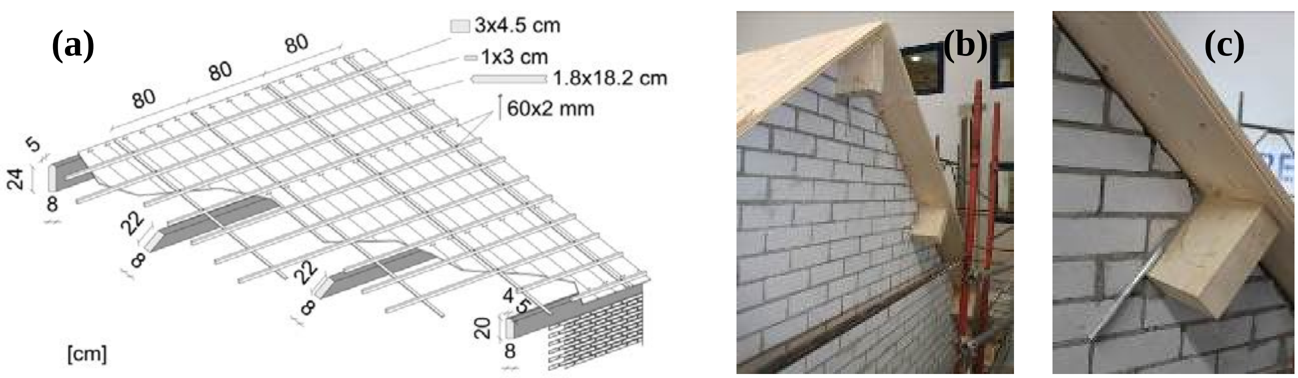


Fig. 6 – Details of the timber roof structure: (a) axonometric view; (b) connection between timber beams and South gable; (c) steel anchors

The roof was completed with the installation of clay tiles. The total mass of the roof, including the weight of the tiles, was 2.8 t. The in-plane roof stiffness was mainly provided by the nailed connections between beams and planks.

2.3 Mechanical properties of materials and components

A testing campaign on material samples, masonry wallettes and structural components was performed in order to fully characterise the mechanical properties of the test-building materials at the Laboratory of the University of Pavia. The sizes of the bricks were respectively 212x102x71 mm for the CS bricks and 211x100x50 mm for the clay bricks. The clay bricks were perforated bricks with ten vertical holes and a void percentage of 17%. They were tested in compression (five CS bricks and five clay bricks) according to EN 772-1. The flexural and compressive strength of the mortar was determined according to the prescriptions of EN 1015-11 [10]. Six wallettes composed of CS and clay masonry, respectively, were tested in compression in the direction perpendicular to horizontal bed-joints according to EN 1052-1 [11]. This allowed the estimation of the compressive strength of the masonry (f_m), and the secant elastic modulus of masonry at the 33% of the compressive strength (E_{m-1}). Bond wrench tests on twenty CS and twenty clay masonry specimens were performed in order to determine the masonry bond strength of the masonry as prescribed by EN 1052-5 [11]. Two fundamental parameters governing the masonry shear strength are the cohesion f_{v0} and the shear friction coefficient of the bed joints μ. Specimens of both masonry typologies were subjected to the shear strength test codified by EN 1052-3 [11]. A parallel testing campaign was conducted at the Delft University of Technology (TU Delft) on specimens built using the same materials [12]. For what concerns the two masonry leaves connection, tests on pull-out strength of this specific coupling system were performed by Messali *et al.* [13]. The tensile ultimate capacity of the steel anchors is approximately 4.3 kN. The concrete used to cast the two slabs had an average compressive strength R_c of 29.5 MPa.

Table 1 – Masonry Mechanical Properties

			Calcium Silicate		***Clay***	
Material property	***Symbol***	***U.M.***	***Average***	***C.o.V.***	***Average***	***C.o.V.***
Density of bricks	ρ_b	*[kg/m³]*	1900	0.02	1650	0.02
Density of masonry	ρ	*[kg/m³]*	1835	0.01	1905	0.03
Compressive strength of bricks	f_b	*[MPa]*	17.18	0.14	30.54	0.17
Compressive strength of mortar	f_c	*[MPa]*	5.4	0.21	6.24	0.09
Flexural strength of mortar	f_t	*[MPa]*	1.71	0.15	1.48	0.19
Masonry compressive strength	f_m	*[MPa]*	5.49	0.1	12.72	0.15
Elastic modulus	E_{m-1}	*[MPa]*	1736	0.26	4742	0.17
Flexural bond strength of masonry	f_w	*[MPa]*	0.056	0.47	0.152	0.65
Masonry (bed joint) initial shear strength	f_{v0}	*[MPa]*	0.03	-	0.11	-
Masonry (bed joint) shear friction coefficient	μ	*[-]*	0.51	-	0.68	-

3. Testing protocol and instrumentation

The specimen was subjected to incremental dynamic test runs, series of table motions of increasing intensity, in order to monitor the evolution of damage at different levels of intensity of shaking. The selected input motions aimed at being representative of expected ground motion in the region of Groningen - a detailed study on the seismic hazard characteristics [14] identified two main scenarios; two smooth-response-spectra records EQ1 and EQ2 with 5-75% significant duration of 0.375 s and 1.72 s and a PGA of 0.095 g and 0.159 g, respectively, were selected to be representative of such scenarios. Fig 7 (left and right) shows the theoretical acceleration time-histories of the experimental inputs and their response spectra, respectively.

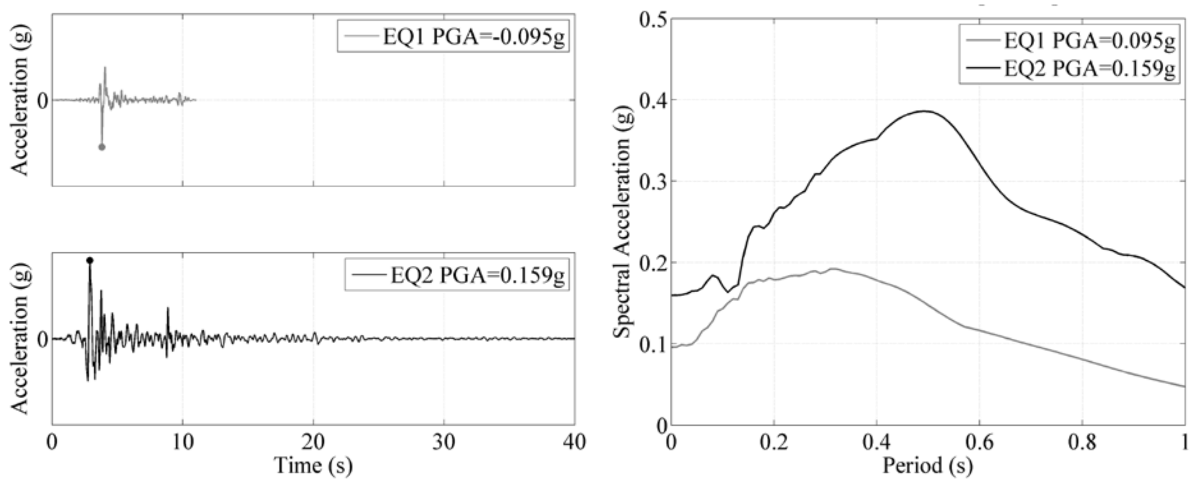


Fig. 7 – EQ1 and EQ2 signals: acceleration time histories (Left) and acceleration response spectra (Right)

Table 2 presents the applied testing sequence specifying the input typology, the intensity and the comparison between the nominal and the recorded PGAs. The table reports values for the recorded peak ground velocities, PGVs, as well. Moreover, some of the tests were preceded by runs of the same typology but with reduced intensity for shaking table calibration purpose (see Table 2, EQ#*). Each test listed in Table 2 was alternated by random noise tests in order to perform structural dynamic identification at each testing step and monitoring the specimen stiffness degradation.

Table 2 – Summary testing sequence

Test Input	*Intensity [%]*	*Nominal PGA [g]*	*Recorded PGA [g]*	*Recorded PGV [m/s]*
EQ1	25%	0.024	0.023	0.015
EQ1	50%	0.051	0.050	0.031
EQ1*	100%	0.102	0.097	0.056
EQ1	150%	0.153	0.138	0.077
EQ2*	50%	0.082	0.085	0.067
EQ2*	100%	0.163	0.166	0.123
EQ2*	125%	0.204	0.192	0.133
EQ2	150%	0.245	0.241	0.164
EQ2	200%	0.326	0.305	0.218

*Shaking table excitations preceded by tests of the same typology but with reduced intensity for shake table calibration purpose

In order to detect and monitor the structural response under different levels of input motion, several instruments were installed on the building. The location and typology of the instrumentation was determined based on the identification of the position of the critical zones and on the physical quantity to be recorded.
The instrumentation consisted of 33 accelerometers and 30 displacement transducers (10 wire and 20 linear potentiometers). The displacements measured between the specimen and the rigid frame were considered equivalent to relative displacements with respect to the shaking table. In particular, wire potentiometers were installed to record the out-of-plane response of the North and South façades at the ground, first-floor and gable mid-height. The linear transducers were instead adopted to monitor directly the longitudinal and transverse displacement of the first and second slabs.

4. Test results

The building sustained a 0.166g PGA event (EQ2 100%) with few visible damage and was considered in near-collapse state after 0.305g PGA (EQ2 200%). The tests were stopped before collapse of the structure, in order to prevent damage to the testing facilities. Videos of the testing sequences are available on EUCENTRE's Youtube channel [15]. The next sections illustrate the qualitative damage evolution as well as the hysteretic responses plotted for each incremental test run.

4.1 Damage evolution

At the end of every stage of the shaking table testing, detailed surveys were carried out for the report of every possible sign of damage having affected the structure. During the testing under the first scenario's seismic excitations (EQ1 150%, 0.138g) the building did not experience any noticeable damage. The specimen suffered only slight damage that became visible just after the test under 100% of EQ2 (0.166g). The formation of a few cracks was observed at the base of the first storey inner-leaf corner piers, associated mainly with their flexural behaviour (corresponding to a 1st inter-storey drift of 0.12% and a 2nd floor displacement of 3.94 mm).

The first cracks being observed in the CS masonry of the second storey were recorded after the test at PGA of 0.241g (150% of EQ2). They are mainly horizontal cracks observed just below the interface between masonry piers and the second floor level slab, as mapped in Fig. 8 (corresponding to a 1st inter-storey drift of 0.34% and a 2nd floor displacement of 9 mm). A horizontal crack was also developed along the base of the squat pier of the second storey, on the West side, indicative of the pier's rocking response. This crack was further extended with a stair-stepped diagonal pattern to the centre of the adjacent spandrel.

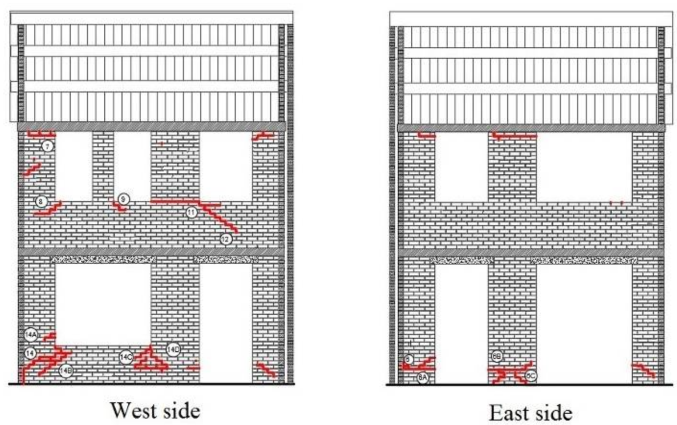


Fig. 8 – Crack pattern in the inner longitudinal walls after the test at nominal PGA of 0.243g (150% of EQ2)

The building experienced a substantial level of damage (compared to that observed under lower intensity shaking) after the test performed at PGA of 0.305g (200% of EQ2). At this shaking level a global response of the structure was triggered, as evidenced by the formation of new cracks or the elongation of pre-existing ones, identified on every specimen' walls, as shown in Fig. 9. Detailed survey of the building was conducted revealing extensive damage in the spandrels of the calcium silicate masonry. In particular, the formation of wide diagonal cracks (starting from the corners of the openings), with sliding of the mortar joints and de-cohesion of blocks were observed (Fig. 10b). In addition, the horizontal cracks located at the top of the second storey piers were extended, reaching a maximum residual sliding of 15 mm (Fig. 10a).

As far as the damage reported in the transversal walls is concerned, the formation of diagonal cracks with an angle of 45° with respect to the horizontal plane, was clearly observed. A stair-stepped cracking pattern through the mortar joints, with opening no greater than 1.2 mm (Fig. 10c). Focusing on the gables, horizontal cracks along their base were apparent (one or two layers above the second floor level), indicative of an out-of-plane overturning mechanism occurring at the gable level. Other cracks are also identified at the locations where the timber beams of the roof are connected with the gable walls. Cracks around these beams are due to interactions of the beams with the supporting masonry gable walls (Fig. 9).

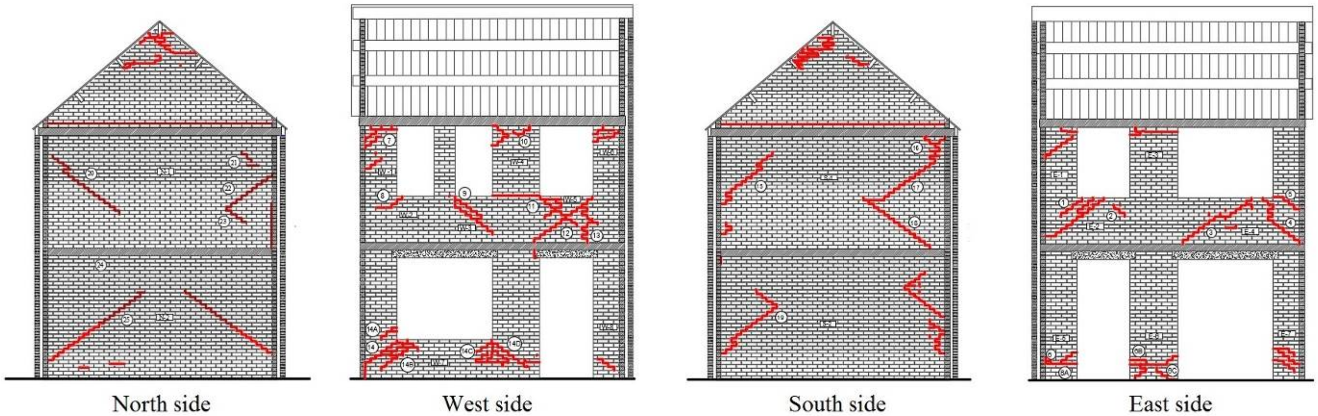


Fig. 9 – Crack pattern in the inner CS walls after the test at nominal PGA of 0.324g (200% of EQ2)

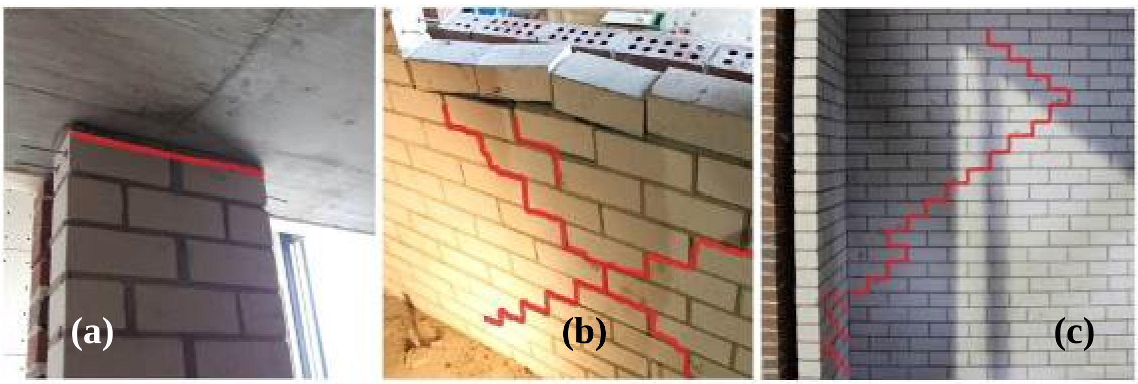


Fig. 10 – Damage observed in the load-bearing masonry: (a) sliding on the top of the second storey longitudinal piers; (b) de-cohesion of masonry blocks and diagonal shear cracks through the joints; (c) stair-stepped cracking pattern through the mortar joints of out-of-plane walls (the cracks are highlighted)

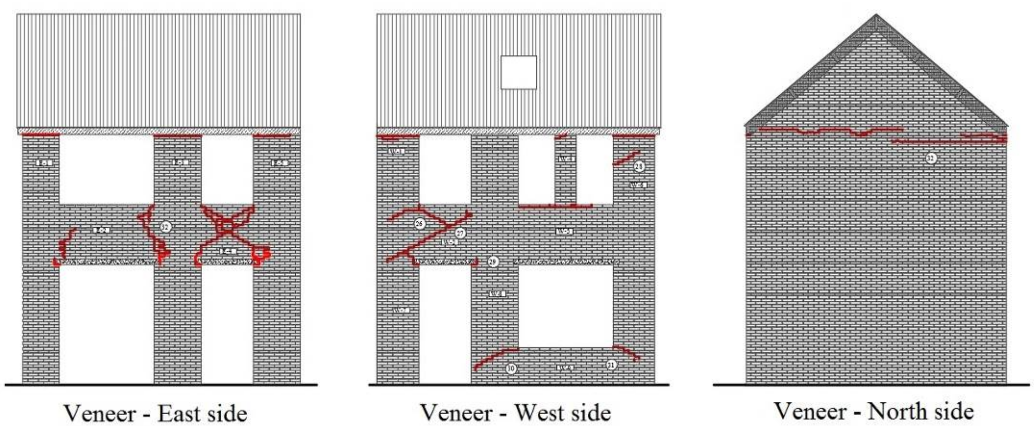


Fig. 11 – Crack pattern in the veneer walls after the test at nominal PGA of 0.324g (200% of EQ2)

Regarding the damage noticed in the veneer walls, perceptible cracks were developed only during the last tests. More specifically, the long spandrel of the eastern façade, developed a flexural mechanism with vertical cracks at both ends originating from the concrete lintels (Fig. 12b), whereas the shorter spandrel presented failure in shear, forming the characteristic X-shape crack pattern (Fig. 12a). On the western side, large stair-stepped shear cracks were observed, such as those crossing the entire short spandrel with an angle of 45°, manifest of a shear failure mechanism.

To a great extent, most of the deformations were absorbed by sliding of the concrete lintels with respect to the masonry supports, as well as sliding at the interface of the roof wall plate timber beams and the second storey masonry piers (Fig. 12c). In the northern veneer, the only cracks observed are located at the second floor level; being extended along the entire length they are associated to the tendency of the gable wall to develop an out-of-plane overturning mechanism (Fig. 11).

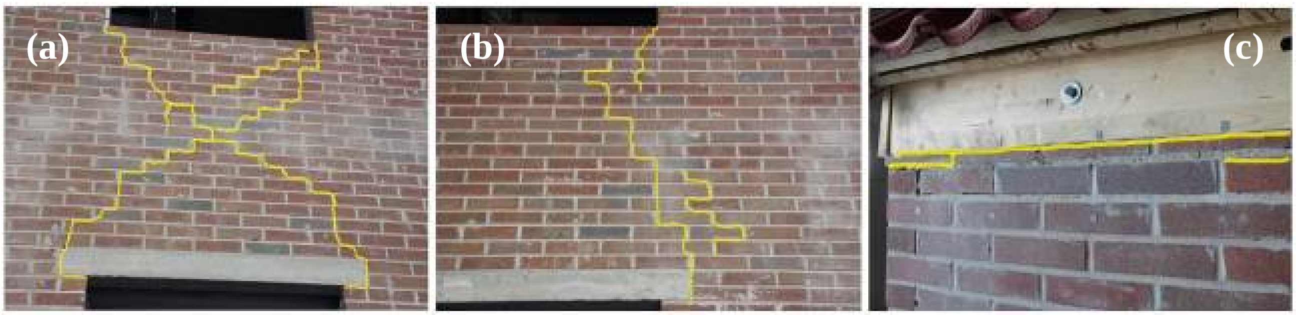


Fig. 12 – Damage observed in the veneer: (a) spandrels failed in shear with the formation of X-shaped crack pattern; (b) development of flexural mechanism evidenced by vertical cracks; (c) sliding at the interface with timber beam

4.2 Hysteretic responses

Fig. 13 shows the evolution of the specimen hysteretic response in terms of base shear vs. second-floor average displacement through all the tests. The base-shear time history has been obtained as the sum of the products of each acceleration history by the tributary mass of the accelerometer. Masses are assumed to be lumped at the accelerometer locations. In grey, the hysteretic response of previous tests is also shown. It is possible to appreciate the progressive specimen stiffness degradation and the consequently fundamental period elongation. It can be observed as the energy dissipation (the area enclosed by the hysteresis loops) is rather high even in the first tests where the specimen is still in the elastic phase and no variation in the fundamental period were observed. The EQ2 input induced a more pronounced asymmetry in the specimen response with respect to the EQ1 earthquake. The displacement and strength demand in the negative direction (towards south direction), indeed, were rather higher than the ones in the positive direction. It is worth noting as a first significant non-linearity in the hysteretic response is detectable during the test EQ2 150% associated with the attainment of spread flexural cracks in the inner CS walls. In the test EQ2 200% with a PGA of 0.305g a large nonlinear behaviour has been observed associated with a diffuse damage on the specimen and a consequently significant reduction of the specimen fundamental period of vibration.

An ultimate inter-storey drift ratio of 0.8-0.9% was achieved for the 1st and 2nd storey, while a value of 1.5% was observed for the significantly more flexible roof structure. The maximum base shear attained was approximately 136 kN (corresponding to a base shear coefficient of 0.249), while the asymmetry depicted in the envelope response curve was a result of the directivity and short duration of the EQ2 accelerogram.

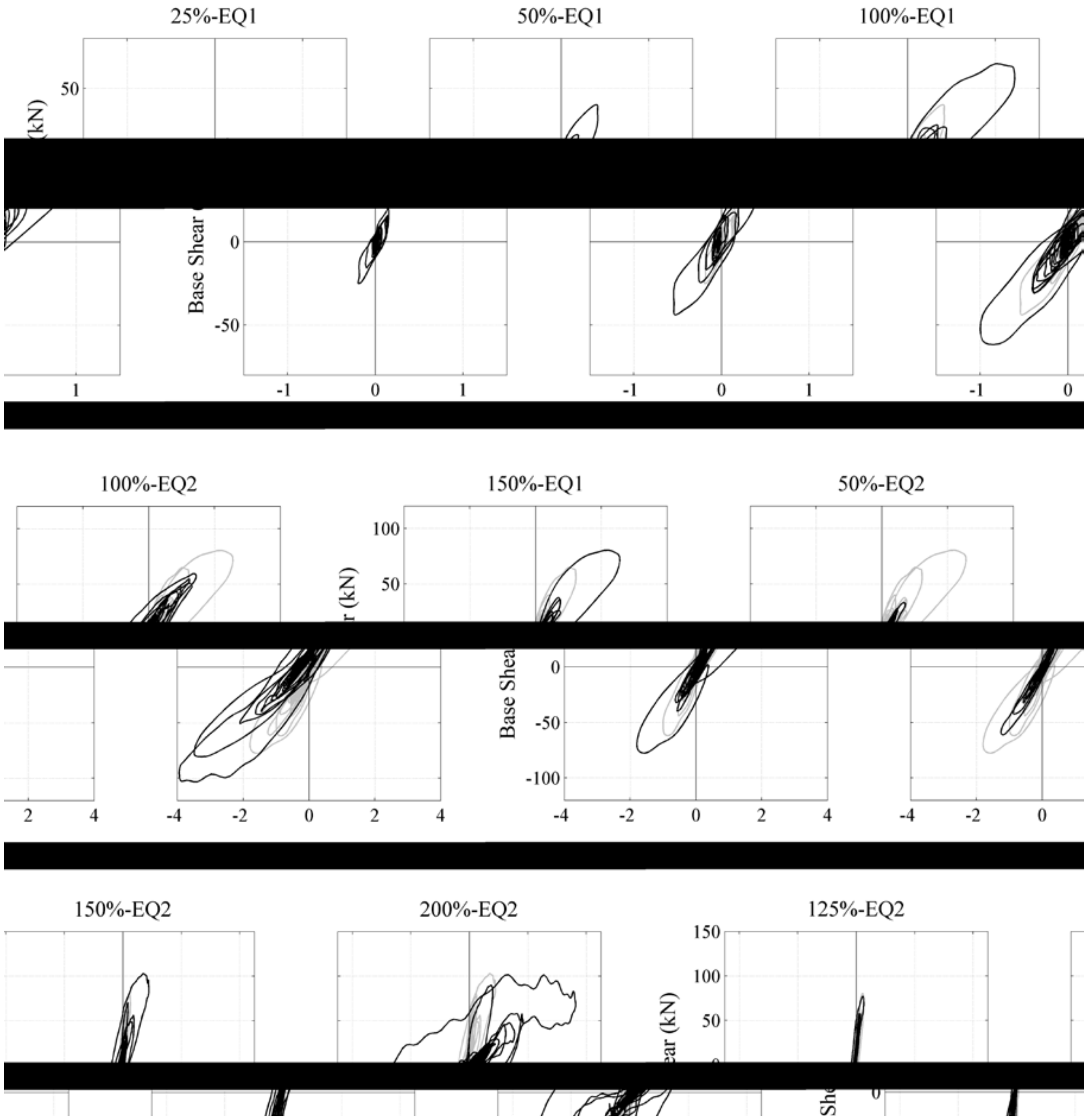


Fig. 13 – Evolution of the hysteretic response (previous tests with respect to the current one are shown in grey)

5. Conclusions

The paper presents a shaking table test performed on a specimen representative of a Dutch terraced house building with cavity walls. The specimen was subjected to incremental input motions representative of two different induced seismicity scenarios characterized by smooth response spectra and a short duration.

The loadbearing structure exhibited a box-type global response thanks to the presence of the rigid concrete slabs, which engaged the longitudinal walls and prevented the occurrence of local out-of-plane failure mechanisms in the transverse walls of the 1st and 2nd stories. As a consequence, the full in-plane capacity of the longitudinal walls was exploited. The building withstood the input motion with a PGA of 0.17g with little damage and was in the near-collapse state at a PGA of 0.31g. No significant shear damage occurred in the masonry piers, which were in general slender, and their response was mainly governed by rocking, whereas sliding occurred at the top of masonry walls parallel to the table motion.

The processing of the recorded signals, both in terms of accelerations and displacements sustained by the tested structure, allowed the evaluation of the seismic resistance and displacement demand at each stage of testing. The large amount of experimental data derived from the full-scale shaking table tests, complemented by a series of tests on smaller structural assemblies and characterization tests on a material level, constitute a useful basis for the development and calibration of numerical models also to reproduce the response of structures with different configurations.

6. Acknowledgements

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