Assessment of seismic vulnerability of non-seismically designed unreinforced masonry buildings through shake table testing

Avaliação da vulnerabilidade sísmica de edifícios em alvenaria simples sem dimensionamento sísmico através de ensaios em mesa sísmica

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Abstract

The response of three full-scale unreinforced masonry buildings, without specific seismic detailing, as observed in shake table testing at LNEC for a seismic action representative of induced seismicity, is presented. The first building specimen simulated the upper floor and roof of the end-unit of a two-storey terraced house, built with unreinforced cavity walls, a reinforced concrete slab and a pitched timber roof. The second specimen was a replica of the roof of the first building. The third specimen represented instead a one-story detached pre-1940's house, built with double-wythe unreinforced clay masonry walls, a timber floor and a steep-pitch roof supported on timber trusses. The specimens were tested up to partial collapse or near-collapse conditions. This paper summarises the main characteristics of the specimens and their experimental results, illustrating their dynamic response and the evolution of damage.

Resumo

Apresenta-se a resposta de três modelos à escala real em alvenaria não reforçada, não dimensionados sismicamente, tal como observada em ensaios na plataforma sísmica do LNEC para uma ação representativa de sismicidade induzida. O primeiro modelo representa o 1º piso e cobertura da unidade final de um conjunto de edifícios em banda, de dois pisos, com paredes de pano duplo, com ligadores metálicos e em alvenaria de tijolo não reforçada, laje em betão armado e cobertura inclinada em madeira. O segundo modelo é uma réplica da cobertura do primeiro. Por outro lado, o terceiro modelo representa uma moradia isolada pré-1940 de um piso, com paredes em alvenaria de tijolo com aparelho holandês, piso em madeira e cobertura de elevada inclinação em madeira. Os modelos foram ensaiados até condições de colapso parcial ou pré-colapso. Este artigo apresenta sucintamente as principais características dos modelos e os resultados experimentais, a sua resposta dinâmica e a evolução do dano.

Keywords: Unreinforced masonry building / Full-scale shake table test / / Induced seismicity / Limit states / Collapse

Palavras-chave: Alvenaria simples / Ensaios em mesa sísmica à escala real / Sismicidade induzida / Estados limites / Colapso

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1 Introduction

This paper discusses the results of three shake table tests, performed at LNEC on full-scale unreinforced masonry (URM) buildings without specific seismic detailing. The geometric and mechanical characteristics of the three prototype buildings, the testing protocol, and the experimental results in terms of damage evolution, response and collapse mechanism are presented. The testing programme was part of a wider research project aimed at assessing the seismic vulnerability of buildings typical of the Groningen region, located in Northeast Netherlands. This area, historically not prone to tectonic ground motions, during the last two decades has been subjected to earthquakes induced by reservoir depletion due to gas extraction [1]. Local structures, not specifically designed for seismic actions, have been exposed to low-intensity shakings, with URM representing almost 90% of the building stock [2].

Because of the limited available information on the seismic performance of Dutch building typologies, an experimental campaign was launched in 2015, aimed at investigating the performance of structural components, assemblies, and systems in pursuance of improving the analytical prediction of URM damage for the vulnerability assessment of URM buildings in the Groningen region. The experimental programme includes *in situ* mechanical characterization tests and laboratory tests, such as: i) characterization tests on bricks, mortar and small masonry assemblies; ii) in-plane cyclic shear-compression tests [3] and dynamic out-of-plane tests on full-scale masonry piers [4]; and iii) full-scale unidirectional and bidirectional shake-table tests on different URM building typologies: cavity-wall terraced houses and substructures [5,6,7] and pre-1940s clay brick detached house [8].

Terraced houses represent more than 50% of the URM building stock of the region under consideration; they are usually two-story buildings with wide openings on two opposite sides, consisting of several side-by-side residential units. Most terraced houses are built with cavity walls, a technology that became popular after World War II, consisting of two leaves of bricks with insulating material sometimes inserted in between. The inner leaf has load-bearing function and is usually made of calcium silicate bricks, while the outer leaf is often a clay-brick veneer with only aesthetic and weather-protection function. The two leaves are typically interconnected by regularly distributed metal ties. Adjacent units of a terraced house are generally separated by double-leaf transverse load-bearing walls, with discontinuous floor slabs resting only on the transverse wall leaves of the individual unit they belong to. Each unit is therefore completely self-supported by transverse walls and structurally independent from the adjacent units. The only shared walls are the outer non-load-bearing veneers. For this reason, only a representative end unit of an entire terraced house was tested.

On the other hand, detached houses constitute another significant portion of the URM building stock of the Groningen region and comprises commonly one- or two-story buildings with irregular plan configurations, wide openings, and flexible floor and roof timber diaphragms. Solid clay-brick walls are generally found in this type of buildings, especially in those preceding World War II. Most detached houses present steep-pitched roofs, with several combinations of external roof shapes and gable geometries.

2 Overview of the specimens

2.1 Cavity-wall terraced house upper portion (CAV-TH-UP)

The first specimen was designed in order to investigate the collapse mechanism of the cavity-wall terraced buildings for both horizontal and vertical excitations. The specimen was a full-scale one-storey building, with a timber roof and RC slab, representing a sub-volume (second floor and roof) of the specimen tested in 2015 at EUCENTRE (CAV-TH). This specimen is called CAV-TH-UP, being the upper portion of a cavity-wall terraced-house building [6]. The building prototype was 5.82 m long, 5.46 m wide and 4.93 m high, with a total mass of 31.7 t.

The RC slab provided rigid diaphragm at the floor and was supported by the transverse (East and West) inner CS leaves, while no gravity load was transferred to the longitudinal (North and South) walls under static conditions. The slab extended over the longitudinal CS leaves but was not directly supported by them: a gap was left between the slab and the longitudinal inner leaves and was filled with mortar only after removal of temporary shoring. There was no direct connection between the longitudinal clay veneers and the slab.

Figure 1 shows the plan view, a picture of the specimen during transportation to the shake table and inner and outer elevations. All the details were identical to the ones of CAV-TH. A rigid steel frame in the interior of the specimen was used as reference system for direct measurement of the floors, walls and roof displacements. A 20-cm-gap in both directions ensured no interference between the slab of the building prototype and the rigid frame.

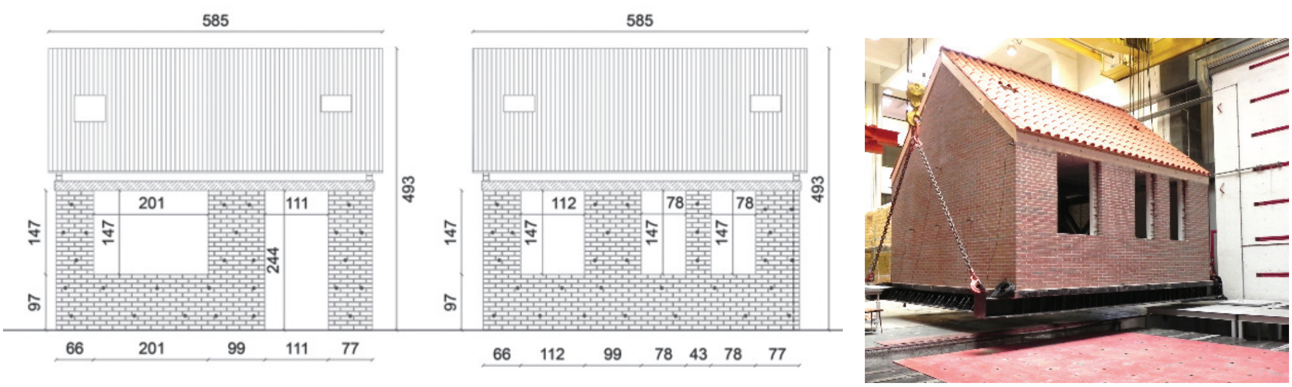
The 42°-pitched timber roof frame included: i) a ridge beam; ii) two wall-plate beams at the base, connected to the slab but not supported by the longitudinal walls; and iii) two girders on each side. Girders and ridge beam were supported by the transverse inner CS leaves, which extended above the floor to form gables. The connection was improved by steel anchors. Timber boards were nailed to the longitudinal framing members. The roof was completed with the installation of clay tiles over a mesh of laths and counter battens nailed to the timber boards.

The roof diaphragm is characterized by four openings (three with dimensions of 54 cm × 45 cm, one of 54 cm × 72 cm) allowing, by means of a cable system, to sustain the RC slab in case of need and preventing a global collapse on the shake table. The in-plane stiffness of the timber diaphragm was essentially provided by the nailed connections between beams and planks, as well as by the effectiveness of the tongue and groove joints.

2.2 Cavity-wall terraced house roof (CAV-TH-RF)

The second test specimen built at the LNEC laboratory, in Lisbon, was a full-scale timber roof with clay tiles, supported on URM gable walls and on a RC slab, representing the roof of the CAV-TH-UP specimen. The East gable wall was made of CS bricks, while the West gable wall was composed of two URM leaves: the inner leaf was also made of CS bricks and the outer leaf was made of clay bricks. The outer leaf was not present in the East façade, simply because the specimen was meant to represent the end-unit of a set of terraced houses.

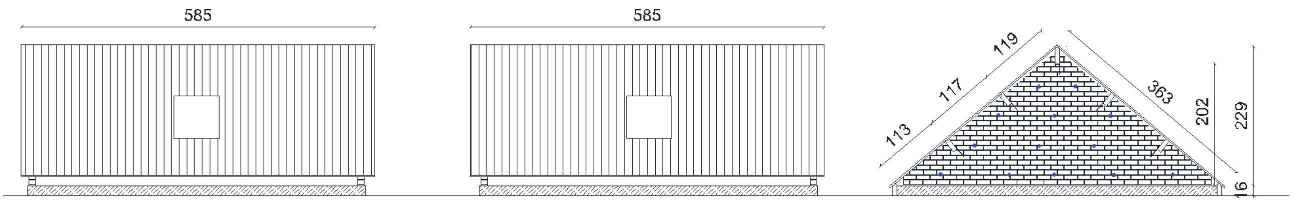
The prototype was tested in the horizontal direction only and it was 5.85 m long, 5.46 m wide and 2.45 m high with a total mass of 17.9 t, of which 11.6 t correspond to the RC slab and 6.3 t to the gable plus roof structure. Figure 2 presents the plan view at the base of the specimen and its North-East and South-West views, while Figure 3 shows the elevation views of the specimen. The blue dots indicate the locations of the steel ties connecting the two leaves. Details of the roof structure are shown in Figure 4.



a) b) c)

Figure 1 Elevations views of the inner CS walls (a, North; b, South) and picture of the specimen during transportation to the shake table (c). Units of cm

Figure 2 Plan view at the base of the specimen and its North-East and South-West views



North Elevation South Elevation West Elevation

Figure 3 Elevation views of the specimen (dimensions in cm)

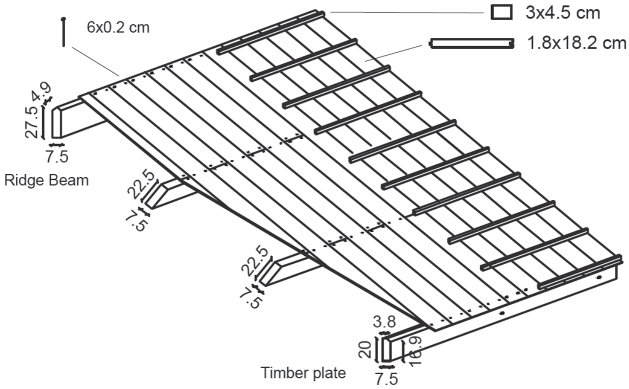


Figure 4 Geometry and details of the timber roof diaphragms (dimensions in cm)

2.3 Clay brick detached house (CLAY-DH)

The prototype building was a single-story house with a 2.72 m floor height and a 2.50 m high symmetrical gambrel roof with tall gable walls, which were weakly connected to the roof framing. Such gables are generally more vulnerable when subjected to out-of-plane excitation; therefore, the shake-table tests were performed in the direction perpendicularly to the gables, as shown by the arrows in Figure 5.

The overall footprint dimensions were 5.66 m in the shaking direction and 5.44 m in the transverse one. The load-bearing structural system consisted of 208 mm thick, double-wythe clay--URM walls in three out of the four perimeter walls. The East façade, built orthogonal to the shaking direction, was made of a single, 100 mm thick wythe wall, with openings both in the first story and the roof. Large asymmetrical openings were also present on

a) b) c)

Figure 5 Full-scale specimen: a) North-West view; b) South-East view; c) ground-floor plan. Units of cm the North and South façades, resulting in varying wall areas in the longitudinal direction, with the intent to magnify differential wall displacements under uniaxial seismic excitation. A 100 mm thick interior wall was built parallel to the direction of shaking, longwise the centerline of the building plan. The wall was 1.98 m long, including two symmetric 0.75 m wide flanges, and did not extend over the floor. The floor was made of timber, resulting in a flexible diaphragm as mostly found in this building typology.

The specimen included two vertical URM chimneys: one was interlocked with the West wall, while the second one was built together with the squat South pier (Figure 2). Both chimneys were designed to have the same flue (34 cm × 34 cm) and a total height of 5.28 m, reaching slightly higher than the roof ridge (5.22 m). The chimney stack in the South façade was sensibly slender, extending approximately 2.3 m above the roofline, while the West stack was squatter as it penetrated the pitched roof very close to the ridge, for about 0.9 m.

2.4 Mechanical properties of materials and components

A mechanical characterization campaign was performed on material samples, masonry wallettes, and structural components. Masonry material properties were obtained following the EN 772, EN 1015, and EN 1052 standards, and the results are summarized in Table 1. The materials employed for the construction of the building specimens exhibit characteristics comparable with those found *in situ*.

Table 1 Material properties

Material property [units]	CAV-TH-UP		CAV-TH-RF		CLAY-DH
	CS	Clay	CS	Clay	Clay
Density of masonry [kg/m^3]	1800	1839	1796	1833	1959
Brick compressive strength [MPa]	18.72	63.23	18.72	63.23	74.20
Mortar compressive strength [MPa]	6.20	8.34	3.70	6.28	2.65
Mortar flexural strength [MPa]	2.87	3.03	2.50	3.67	1.22
Masonry compressive strength [MPa]	9.80	19.39	7.03	16.17	11.45
Masonry Young's modulus [MPa]	7955	12798	6090	12661	9120
Masonry flex. bond strength [MPa]	0.36	0.19	0.33	0.19	0.36
Bed joint initial shear strength [MPa]	0.45	0.41	–	–	0.47
Bed joint shear friction coefficient [-]	0.48	0.75	–	–	0.81

3 Instrumentation and testing protocol

Several sensors were installed on the buildings to monitor their structural response. The instrumentation of the CAV-TH-UP specimen consisted of 40 accelerometers, 8 wire potentiometers and 16 LVDTs; the CAV-TH-RF specimen was instrumented with 28 accelerometers, 9 wire potentiometers, and 12 LVDTs; the CLAY-DH specimen was instead equipped with 40 accelerometers, 8 wire potentiometers and 16 LVDTs. A rigid steel-frame was installed in the interior of the specimens as a reference system for direct measurements of the displacements.

The CAV-TH-UP specimen was subjected to incremental bidirectional dynamic tests (longitudinal and vertical), applying a series of shake-table motions of increasing intensity on the LNEC shake table, while the other specimens were subjected to longitudinal motions only. A study identified two records compatible with the smooth-response-spectra shown in Figure 6 [9]. The acceleration amplitude of these two records was then scaled in order to obtain the desired incremental test protocol, summarized in Table 2. These tests were conducted up to the near-collapse state of the specimens.

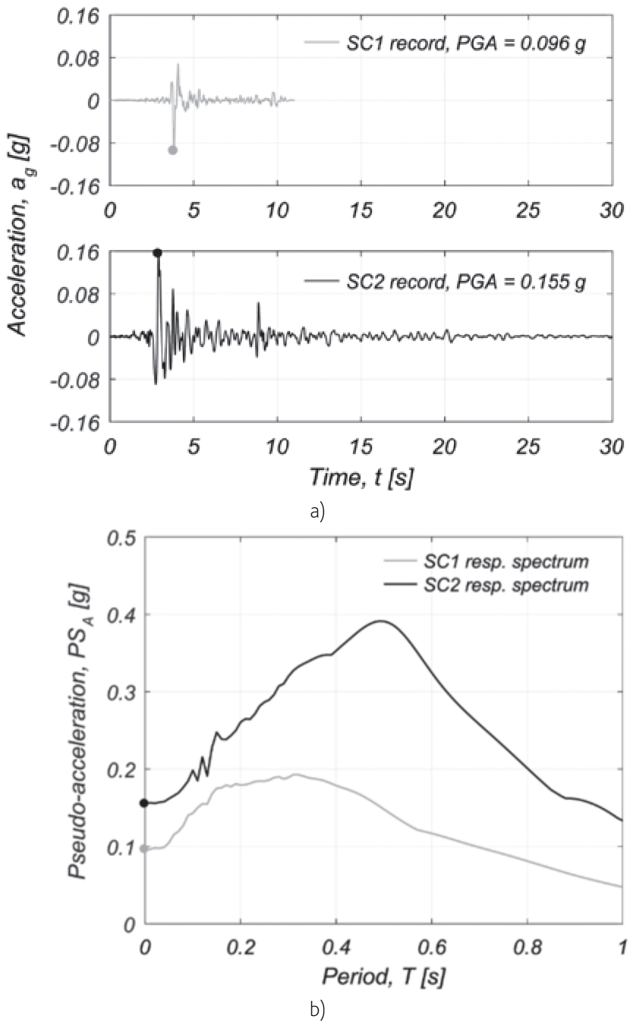


Figure 6 SC1 and SC2 signals: a) acceleration histories; b) elastic pseudo-acceleration response spectra for 5% viscous damping ratio

The CAV-TH-UP and CAV-TH-RF specimens represent the upper part of a terraced-house unit. For this reason, the acceleration histories replicated the first-floor motion, and second-floor, respectively, of a terraced-house subjected to SC1 and SC2. The horizontal motions were the ones recorded during previous tests on a 2-story model [5], while the vertical accelerograms were compatible with the two scenarios and directly applied at the shake table, *i.e.* considering the missing first floor as rigid in the vertical direction. The applied spectra reflect the frequency content due to the damage evolution, enlarging the quasi-constant acceleration plateau.

4 Test results

4.1 Damage evolution and collapse mechanism of CAV-TH-UP

In this test, the shake table motions are named FEQ1 and FEQ2 instead of SC1 and SC2, for distinction between the first-floor motion and the ground one. The first damage (crack width of 0.2 mm) associated with a shake-table motion appeared on the plaster layer of the spandrel between piers 5 and 6 in the South CS wall, during test FEQ1-100% (H-PTA = 0.12 g).

Table 2 Summary of the testing sequence

Test input	Scale factor [%]	Nominal PGA [g]	Recorded PTA [g]			
			CAV-TH-UP		CAV-TH-RF	CLAY-DH
			Hor.	Vert.		
SC1	50	0.048	0.056	0.036	0.074	0.050
SC1	100	0.096	0.119	0.075	0.14	0.099
SC1	150	0.14	0.146	0.122	0.17	0.13
SC2	50	0.077	0.095	0.071	0.106	0.087
SC2	100	0.16	0.218	0.100	0.207	0.16
SC2	150	0.23	0.380	0.214	0.245	0.21
SC2	200	0.31	0.393	0.184	0.487	0.29
SC2	250	0.39	–	–	–	0.47
SC2	300	0.46	0.630	0.343	0.668	0.58
SC2	400	0.62	–	–	0.935	0.68
SC2	500	0.77	–	–	0.955	1.0

PGA = reference peak ground acceleration.
PTA = peak table acceleration.

During test FEQ1-150% (horizontal peak table acceleration, H-PTA = 0.15 g), two horizontal cracks with a negligible width developed at the base of the CS piers 4 and 6, associated with the activation of a flexural/rocking response. The crack at the base of pier 4 continued horizontally, for a length of approximately 1 m, in the transverse CS wall of the West side, probably due to a sort of flange effect. The observed damage did not change during test FEQ2-50% (H-PTA = 0.1 g). A similar crack, due to a flexural/rocking behavior, was surveyed at the base of pier 1 of the North wall at the end of test FEQ2-100% (H-PTA = 0.22 g).

The FEQ2-150% test (H-PTA = 0.38 g) caused the development of new cracks and the elongation and widening of the pre-existing ones; a 1-mm-wide stepped diagonal crack appeared on the spandrel between piers 1 and 2 of the North CS wall. A further worsening of the existing crack pattern has been noticed at the end of test FEQ2-200% (H-PTA = 0.39 g); new horizontal cracks with negligible thickness have been observed in the plaster of pier 5 and spandrel between piers 4 and 5 of the South CS wall. Despite a clear rocking response of the gable walls, evident from the displacement histories recorded by the displacement transducers, no visible cracks were detected on them at the end of the tests. Moreover, the clay veneer walls did not suffer any significant visible damage up to test FEQ2-200%.

The (partial) collapse of the specimen prototype was attained during test FEQ2-300% (H-PTA = 0.63 g), exhibiting a rather fragile behavior. The pronounced rocking mechanism developed by the slender longitudinal piers and the vertical input motion on the prototype led to an uplift of the RC slab, causing a loss of restraint at the top of the East CS transverse wall, which failed out-of-plane. In addition to this local failure mode, a global severe damage in all longitudinal piers, associated with expulsion of materials, was observed. Severe damage occurred also in the West CS transverse wall due to the interaction between the displacement drift imposed by the floor and the out-of-plane actions induced by the wall's inertial forces and the outer veneer wall (*e.g.* pushing and pulling the wall by means of the steel ties). Cracks have been observed on both East and West CS gable walls, in particular in the regions close to the L-shaped steel anchors.

Regarding the performance of the outer veneer wall, no significant damage has been observed up to test FEQ-300%. Large relative displacements between the two leaves were measured (similarly to CAV-TH), even at earlier stages of testing. During the test where the specimen reached a partial collapse, horizontal cracks have been surveyed at the base of all longitudinal piers, a clear sign of rocking/sliding behavior. The crack pattern surveyed on the West wall shows both the rocking mechanism of the system gable walls and roof and the triggering of a global pull-out/push-in of the veneer wall which is not directly connected to the slab.

Figure 8 plots the backbone curve in terms of base-shear coefficient (BSC) and global drift ratio for the specimen. The global drift ratio is the upper floor displacement divided by the corresponding height above the shake table. The BSC was computed as the sum of the products of each accelerometer's readings times its tributary mass, lumped at the accelerometer location, divided by the total weight of the specimen. The (partial) collapse of CAV-TH-UP was reached at a global drift of more than 4%, under a PGA of 0.48 g, with the other damage states identified in Figure 8.

4.2 Damage evolution and collapse mechanism of CAV-TH-RF

The outside of the East CS wall was covered with a white plaster layer, making the detection of new cracks easier. Figure 9 shows the evolution of the damage surveyed on both gable walls throughout the entire testing sequence.

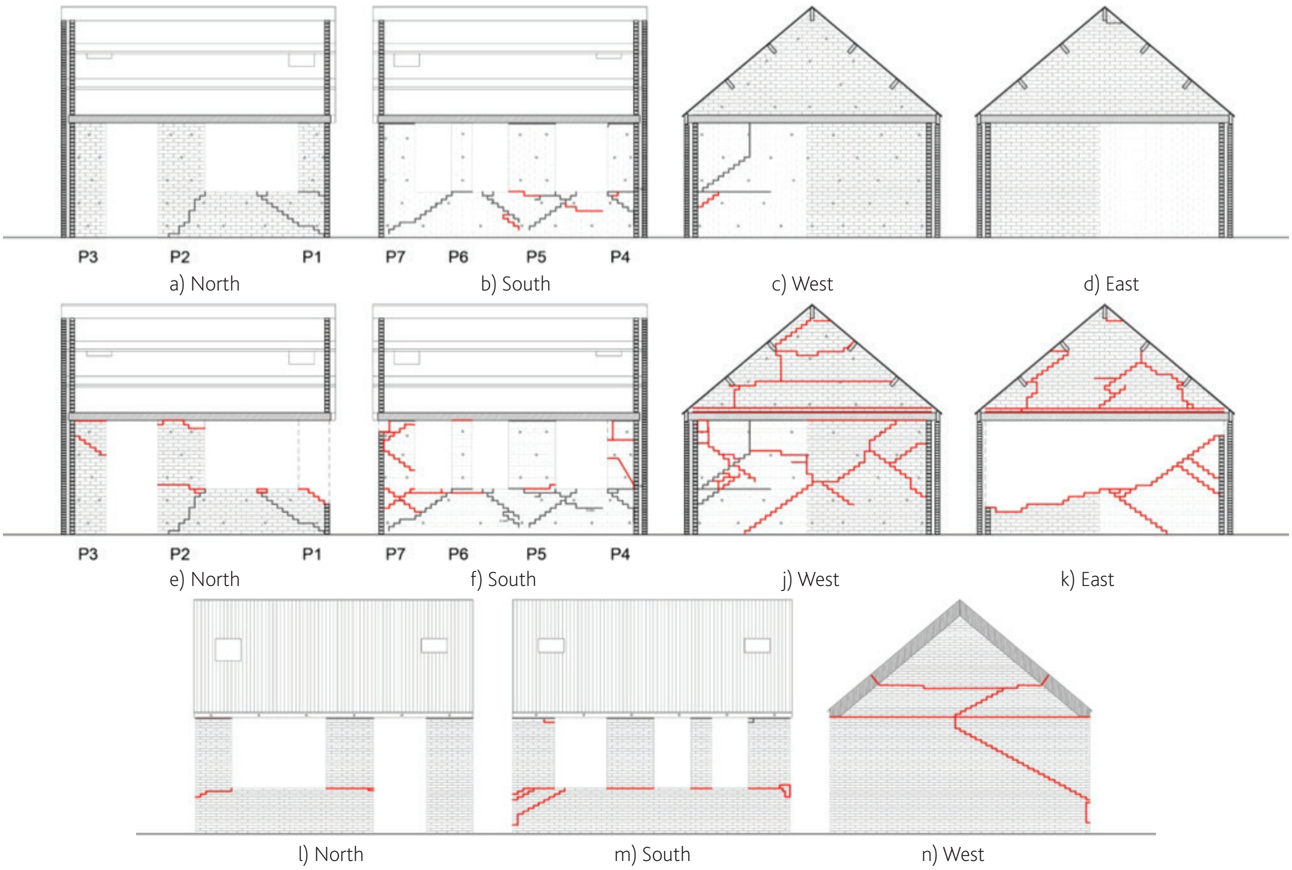


Figure 7 Crack pattern on the CAV-TH-UP specimen after: (a-d) test FEQ2-200%, inner walls; (e-k) test FEQ2-300%, inner walls; (l-n) test FEQ2-300%, veneers. Cracks marked in black were pre-existing. Lightly shaded areas on the inner wall sketches identify plastered areas

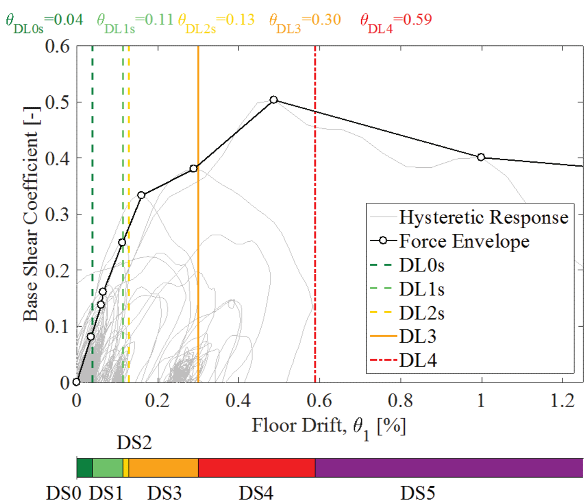


Figure 8 Base shear backbone curve and damage state limits on the CAV-TH-UP specimen

The first visible damage associated with a shake table motion was detected during test FEQ1-100%. Minor cracking was observed on the East wall, around the L-shaped steel anchors connecting the CS wall to the timber roof beams. This was a very minor damage, only visible on the plastered wall and not represented in Figure 9. No particular additional damage was visible during tests FEQ1-150% and FEQ2-50%, although a slight reduction of the specimen's fundamental frequency of vibration was detected.

There was a crack opening at the base the CS East wall during test FEQ2-100%, with a permanent crack width of around 0.1 mm, due to a clear rocking response in that wall. Despite no crack being visible on the inner CS West wall, a coupled rocking response was measured between the two leaves and a crack was observed at the outer clay wall. Test FEQ2-150% caused no new damage on the structure, while FEQ2-200% has only extended already existing cracks. Nevertheless, a new significant reduction of the fundamental frequency of the roof specimen was observed after FEQ2-200%, similar to the reduction observed after FEQ2-100%.

During test FEQ2-300%, several new sub-horizontal cracks formed on the East gable wall with their origin at the connections between the CS wall and the roof beams. No new cracks were identified on the West wall for this test, nor for the following one: FEQ2-400%. On the other hand, the latter test induced an enlargement of the

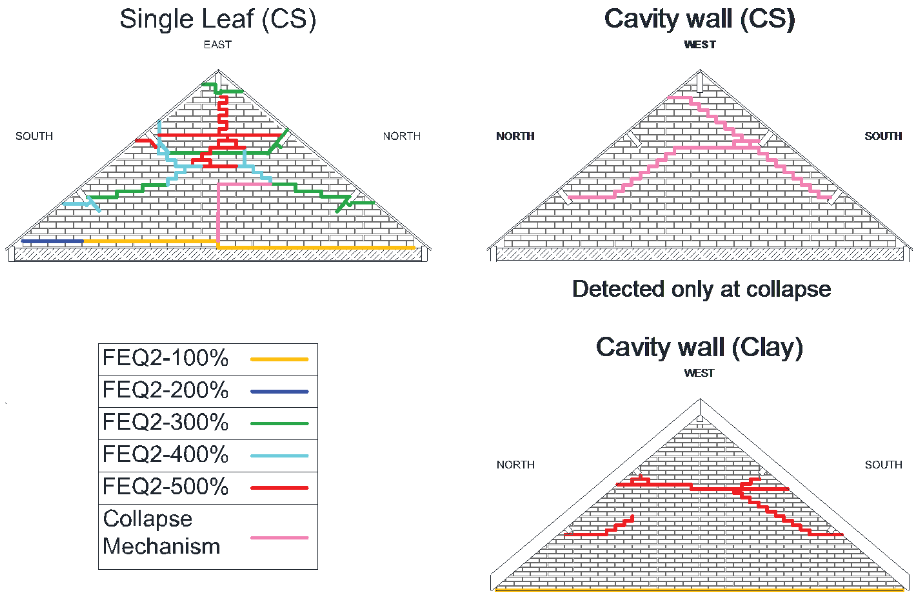


Figure 9 Evolution of the crack pattern in the gable walls of the CAV-TH-RF specimen

Figure 10 Damaged specimen at the end of test FEQ2-600%

cracks on the East wall, interconnecting several of the pre-existing ones. At this point, several instruments were removed.

Afterwards, test stage FEQ2-500% generated a set of cracks on the outer clay leaf of the West wall, very similar to the one produced on the East wall during FEQ2-300%. On the East wall, the main crack opening was a vertical one from the ridge beam downwards, largely contributing to the formation of the collapse mechanism mobilized on the subsequent test, FEQ2-600%. During this last test, another vertical crack formed on the East gable wall, now originating from the bottom of the wall and completing its collapse mechanism. Only then important cracks on the inner leaf CS West wall were detected. The (partial) collapse of the specimen prototype was thus attained during test FEQ2-600%. Figure 10 illustrates the final damaged state of the model and the unrecovered permanent deformations. It is especially interesting to note that, even at this post-collapse state, the West wall and the timber roof system still retained a full load-carrying capacity for gravity loads.

The evolution of the specimen's backbone response is shown in Figure 11, in terms of BSC *versus* roof diaphragm drift, γ_R. The identification of global quantitative thresholds that adequately describe the overall structural damage state of the building is also attempted in Figure 11.

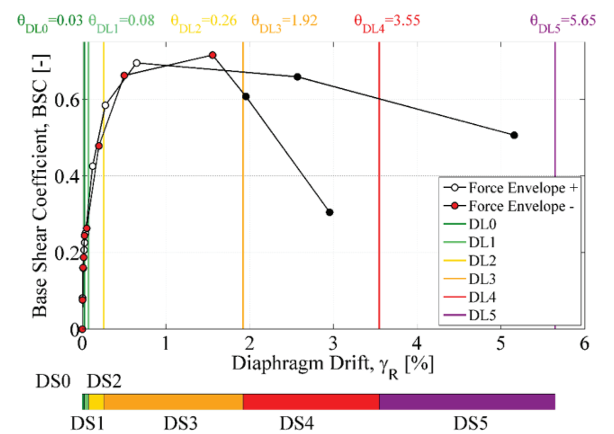


Figure 11 Base shear backbone curve and damage state limits on the CAV-TH-RF specimen

4.3 Damage evolution and collapse mechanism of CLAY-DH

The building specimen did not suffer any visible damage up to test SC2-150% (PGA = 0.21 g), began showing minor cracks for shaking under SC2-200% (PGA = 0.29 g), and was considered at near-collapse state after test SC2-400% (PGA = 0.68 g) when the West chimney collapsed, and the rest of the structure underwent substantial degradation. During test SC2-500% (PGA = 1.0 g), debris from the West chimney fell in the interior of the building and

Figure 12 Observed damage after test SC2-500%: a) collapse of West façade chimney; b) collapse of the West chimney in the interior of the building; c) flexural crack at the base of a North pier; d) cracks at the top of the North wall due to out-of-plane mechanism of the East façade; e) large permanent openings on the North spandrels; f) cracking of the South chimney; g) mortar-joint sliding on the West façade; h) near-collapse state of the East façade; i) damage in the interior wall; j) horizontal crack at the base of the South chimney stack

portions of the East and North façades displaced as rigid bodies by sliding. A considerable percentage of the walls had lost their load-bearing capacity, and the structure was barely in equilibrium. The building would not survive further shaking; therefore, tests were stopped to prevent collateral damage to the instrumentation and the shake table.

Figure 12 illustrates the damage appeared on the building by the end of the testing sequence. A detailed description of the damage evolution can be found in [8]. Among other aspects, the tests allowed defining damage limit states, from first structural damage up to near-collapse conditions, for the performance-based assessment of clay-URM buildings, as shown in Figure 13.

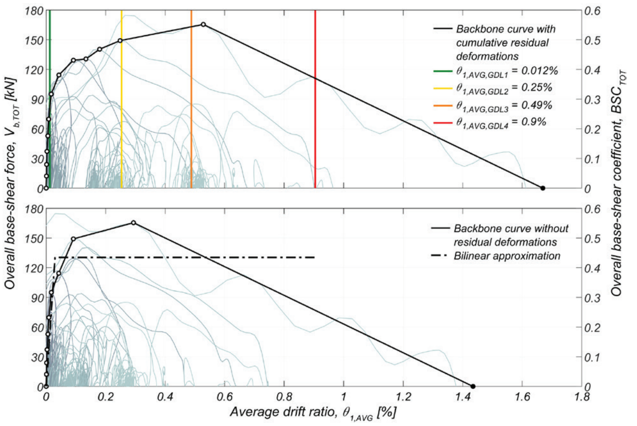


Figure 13 Base shear backbone curve and damage state limits on the CLAY-DH specimen

5 Conclusions

This paper discussed the seismic vulnerability of non-seismically designed URM buildings based on full-scale shake-table tests of two building specimens simulating a Dutch terraced house building with cavity walls (two different tests) and a clay-brick detached house. The specimens were subjected to incremental input motions representative of induced seismicity scenarios for the Groningen region in the Netherlands, characterized by smooth response spectra and short significant durations. A description of the damage evolution, of the degradation of dynamic properties, and of the hysteretic response of the specimen during the shake table tests was provided, as well as the identification of damage state limits.

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