

Experimental seismic performance of a full-scale unreinforced clay-masonry building with flexible timber diaphragms

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ABSTRACT

This paper presents the results of a unidirectional shake-table test performed on a full-scale, single-storey unreinforced masonry building. The specimen represented a typical detached house of the Groningen region of the Netherlands, consisting of double-wythe clay-brick unreinforced masonry walls, without any specific seismic detailing. The building prototype included large openings and a reentrant corner, causing a discontinuity in one of the perimeter walls. The floor was made of timber beams and planks, resulting in a flexible diaphragm. The roof, characterized by a very steep pitch, consisted of a series of timber trusses connected by wood purlins and boards. The two façades perpendicular to the shaking direction were designed to represent two typical gable geometries. An incremental dynamic test was conducted up to the near-collapse state of the specimen, using input ground motions compatible with induced-seismicity scenarios for the examined region. This paper summarizes the main characteristics of the specimen and the shake-table experimental results, illustrating the dynamic response of the structure, the evolution of the damage mechanisms, and the attainment of significant limit states.

1. Introduction

The Groningen region of the northern Netherlands, historically not prone to tectonic earthquakes, in recent years has been subjected to seismic events induced by gas extraction and consequent reservoir depletion [1]. Local structures, not specifically designed for seismic actions, have been exposed to low-intensity shakings during this period, with unreinforced masonry (URM) buildings 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 analytical prediction of URM damage for the vulnerability assessment of URM buildings in the Groningen region. The experimental program includes in-situ mechanical characterization tests [3] and laboratory tests, such as: (i) characterization tests on bricks, mortar and small masonry assemblies; (ii) in-plane cyclic shear-compression tests [4] and dynamic out-of-plane tests on full-scale masonry piers [5]; and (iii) full-scale unidirectional and bidirectional shake table tests on different

URM building typologies [6–9].

With the aim of investigating the seismic behaviour of clay-brick URM detached houses dating back to before World War II, an incremental dynamic test was carried out on a prototype building at the EUCENTRE laboratory in Pavia, Italy, up to near collapse conditions. This typology constitutes a significant portion of the URM building stock of the Groningen region and comprises commonly one- or two-storey buildings with solid clay-brick walls, irregular plan configurations, wide openings, and flexible floor and roof diaphragms. Most detached houses are characterized by steep pitched roofs, with several combinations of external roof shapes and gable geometries [10,11].

The building specimen was dynamically excited in the direction perpendicular to roof trusses and floor beams, which is considered more vulnerable. The opening sizes and locations resulted in different shear stiffness and strength for the two parallel shear walls. Ground shaking of increasing intensity was applied to the building base, while random vibration tests were performed to monitor the evolution of the system dynamic properties at each testing step. The specimen was densely instrumented with sensors that recorded the response of various structural elements. Among other aspects, the experiments allowed to

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investigate: (i) the load sharing and possible torsional effects, if any, between the two longitudinal walls, (ii) the out-of-plane behaviour of the gables, (iii) the degree of connection between walls and roof or floor framing systems, and (iv) the in-plane flexibility of roof and floor diaphragms, which mostly affect the seismic vulnerability of these structures.

This paper presents the geometric and mechanical characteristics of the building specimen, the construction details, the instrumentation, the seismic input and the testing protocol, and the major observations from the tests including damage evolution and hysteretic response. The general behaviour of the prototype building is discussed and its performance is assessed by linking engineering demand parameters, performance limits states, and points of the global force-displacement response.

2. Specimen overview

2.1. Specimen geometry

The full-scale single-storey prototype was characterized by a 2.9-m floor height (measured to the top of the attic floorboards) and a 3.3-m-high pitched roof. The overall footprint dimensions were 5.8 m in the shaking direction and 5.3 m in the transverse one. The load-bearing structural system consisted of 208-mm-thick, double-wythe clay URM walls, supported by a composite steel-concrete foundation rigidly fixed to the shake table. The specimen included large asymmetrical openings on all sides and a reentrant corner, causing a discontinuity in one of the perimeter walls (Figs. 1 and 2), with the intent to magnify differential wall displacements under uniaxial seismic excitation compared to a more regular rectangular layout.

The timber floor and roof diaphragms were flexible, as mostly found in this building typology. The roof external shape was designed to combine two different end geometries: a half-hipped roof with clipped gable at the North façade and a full-height gable at the South façade (Figs. 1 and 2). The perimeter walls extended above the first floor to form 208-mm-thick gables. These elements are generally more vulnerable when subjected to out-of-plane excitation because of weak connections to the roof framing along this direction. For this reason, the unidirectional shake table test was performed perpendicularly to the gables, as shown by the arrows on Fig. 1.

Even though not expected to be exhaustive of all possible geometric variations of the local building stock, the building prototype was deemed representative of a pre-1940s clay-brick URM detached house of the Groningen region (Fig. 1a and b). Contractors from the Groningen area built the specimen, using materials shipped from the Netherlands. The specimen was built at full scale directly on the shake-table of the EUCENTRE laboratory, to avoid possible damage during transportation.

2.2. Construction details

Construction details of the Dutch practice preceding the 1940s were reproduced in the specimen. The *Dutch cross* brickwork bond (Fig. 3a) was adopted for the masonry bearing walls, with $208 \times 100 \times 50$ mm solid clay bricks and 10-mm-thick, fully mortared head and bed joints. Lintels were built above all openings (Fig. 3b, and c): they consisted of a 100-mm-wide $\times$ 50-mm-deep timber beam below the interior masonry wythe, extending into the masonry 100 mm on each side of the opening for support. A 300-mm-deep brick flat arch was built below the exterior masonry wythe, with the brick stretchers facing outwards.

The floor system consisted of 200-mm-wide $\times$ 24-mm-thick spruce timber floorboards, nailed perpendicularly to ten 80-mm-wide $\times$ 180-mm-deep timber joists spanning continuously between the East and West URM walls (Fig. 4). The joist ends were cut at an 80° angle (Fig. 5) and were supported on the interior wythe of the longitudinal walls at a height of 2.7 m above ground level.

Connection between the floor diaphragm and the East and West walls was provided by 14-mm-diameter L-shaped steel anchors (labelled X1 on Figs. 4a and 5), screwed to the timber joists and embedded into the masonry between the two wythes (Fig. 5a, b, d, and e). Flat S-shaped steel connectors (labelled Y1 on Figs. 4a and 5) provided wall-to-diaphragm connection for the North and South façades, restraining these walls against out-of-plane overturning. These anchors, located at mid-span of each façade (Fig. 4a), ran below the timber floorboards and were screwed to the first two floor joists from the restrained walls (Fig. 5c, and f). Additional S-shaped and L-shaped anchors were installed at the gable-floor and gable-roof joints, respectively: these anchors were not connected to the timber framing initially, but were placed with the possibility to be nailed subsequently in case of premature development of local out-of-plane mechanisms of the gables. Since they were never activated, the additional anchors are not represented on Fig. 4c.

The roof structure consisted of four East-West timber trusses, supporting longitudinal North-South purlins and a ridge beam (Fig. 6). The truss rafters were connected to timber wall plates above the longitudinal East and West walls and above the North clipped gable. The longitudinal wall plates were screwed to a series of gutter beams recessed into the masonry, and placed above a mortar layer (Fig. 7b). At the North roof-gable interface the wall plate was nailed to three gutter beams, without mortar above the bricks (Fig. 7c); this configuration was expected to accommodate relative displacements between the roof and the top of the clipped gable.

A truss was placed back-to-back with the South gable; however, the roof purlins extended through and protruded 100 mm beyond the masonry gable (Fig. 7d). This resulted in a very small fraction of gravity load being transmitted to the South gable under static conditions. Two planks were nailed to the purlins outside the gable (Fig. 7e), forming an L-shaped end-block which restrained the relative displacement between

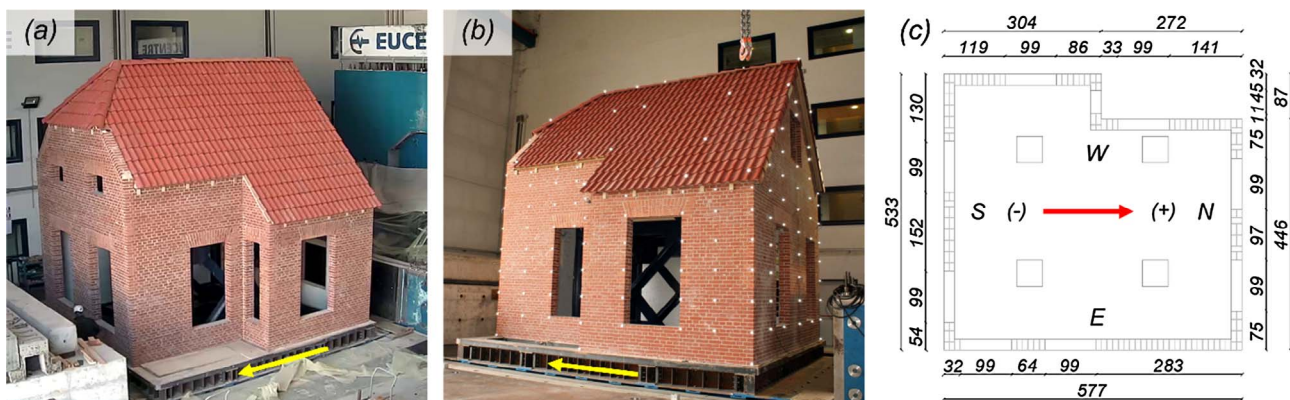


Fig. 1. Full-scale specimen: (a) North-West view; (b) South-West view; (c) first-floor plan (units of cm).

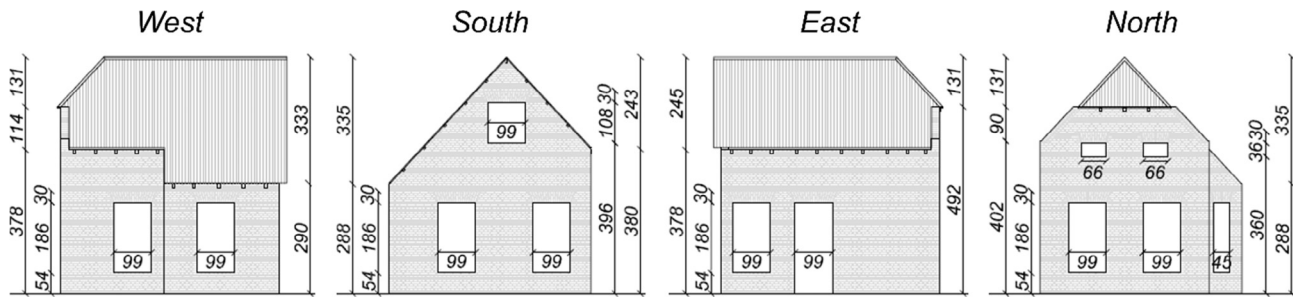


Fig. 2. Elevation views of the specimen (units of cm).

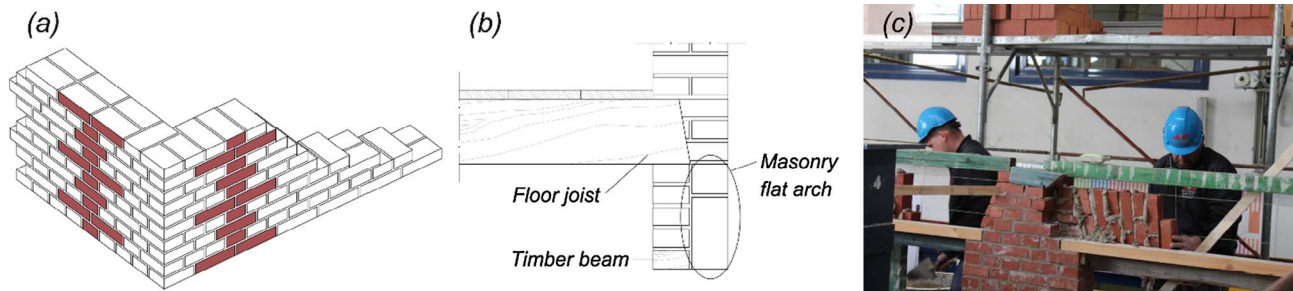


Fig. 3. Construction details of the test-building: (a) Dutch cross bond scheme; (b) section of a lintel; (c) construction of the lintels.



Fig. 4. First-floor framing: (a) floor framing plan with wall-to-diaphragm connections; (b) floor joists during construction; (c) floorboards during construction (units of cm).

gable and roof due to gable out-of-plane response. 18-mm-thick $\times$ 200-mm-wide timber boards were nailed perpendicularly to the purlins above the roof framing. The roof was completed with clay tiles, supported by a mesh of laths and counter battens nailed above the timber boards (Fig. 7f).

A rigid steel frame was installed inside the building specimen. This structure served as a safety system, providing support in case of partial or global collapse of the specimen, and constituted a rigid reference system for direct measurement of floor, walls and roof displacements. The frame was not in contact with the building: its columns ran through four holes in the floor diaphragm, oversized to accommodate the specimen lateral displacements (visible in Figs. 4 and 7a). A video showing the construction phases of the building specimen is available on the EUCENTRE YouTube channel [12].

2.3. Masses

The masonry had a mean density of 1984 kg/m^3 . Masonry walls, floor diaphragm, and finished roof provided masses of 29.0 t, 0.47 t and 1.87 t, respectively. An additional mass of 1.31 t was provided to the first floor by laminated rubber blocks, evenly distributed over the diaphragm to account for superimposed dead and live loads. The total

mass of the building specimen resulted to be 32.6 t (Table 1).

2.4. Mechanical properties of materials and components

A mechanical characterization campaign was conducted on the clay-brick masonry of the building prototype at the DICAr laboratory of the University of Pavia. The testing series comprised strength tests on clay units (Fig. 8a and b) and mortar samples (Fig. 8c and d), as well as compression and bending tests on small masonry assemblies (Fig. 8e, f, and g), bond wrench tests (Fig. 8h) and triplet tests (Fig. 8i).

The solid clay bricks had compressive strength $f_b = 47 \text{ MPa}$ and flexural-tensile strength $f_{bt} = 8.5 \text{ MPa}$ [13]. The compressive and flexural-tensile strength of the mortar were $f_c = 4.12 \text{ MPa}$ and $f_t = 1.2 \text{ MPa}$, respectively [14]. Six masonry wallettes were tested in compression perpendicularly to the horizontal bed-joints [15], allowing an estimation of the masonry compressive strength, $f_m = 9.23 \text{ MPa}$, and elastic modulus secant at 33% of the compressive strength, $E_{m1} = 8123 \text{ MPa}$. Four-point in-plane and out-of-plane bending tests were performed on six and five masonry wallettes, respectively, in order to evaluate the in-plane ($f_{x3} = 0.44 \text{ MPa}$) and out-of-plane ($f_{x2} = 0.64 \text{ MPa}$) masonry flexural strengths [16]. Bond wrench tests were performed on eighteen specimens [17] to determine the bond

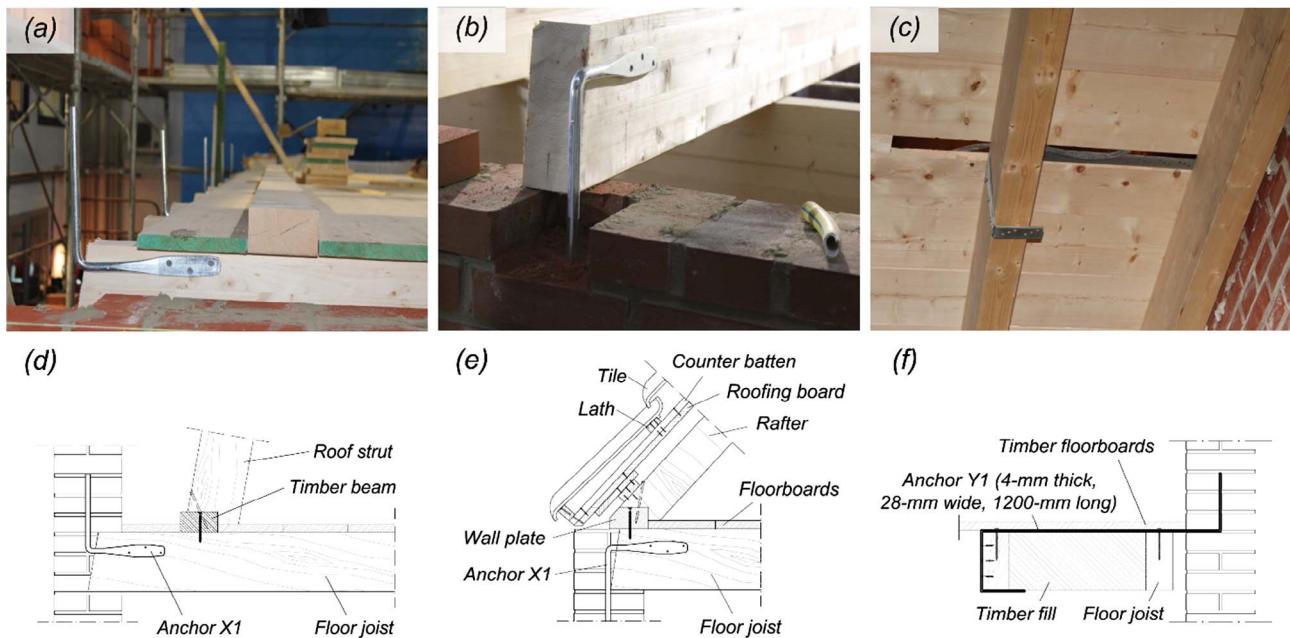


Fig. 5. Wall-to-diaphragm connections: (a, d) East wall; (b, e) West wall; (c, f) North and South walls.

strength of masonry, $f_w = 0.23$ MPa. Masonry triplets were subjected to shear tests [18] to determine the masonry bed-joint cohesion, $f_{v0} = 0.15$ MPa, and shear friction coefficient, $\mu = 0.55$. The results of all complementary tests performed on materials for the building prototype are summarized in Table 2.

An in situ testing campaign was conducted in 2015, to evaluate ranges of material properties characterizing the URM building stock of Groningen. The mechanical properties were determined from both non-destructive methods, such as rebound hammer tests on brick units and penetrometric tests on mortar joints, as well as partially-destructive techniques, including flat-jack and shove tests. Table 2 compares average values and dispersions obtained for the building prototype (representing a variability of the masonry properties within the specimen, i.e. intra-building variability), with the estimates from the in-situ tests on pre-1940s clay-brick masonry buildings. The in-situ data (mean and C.o.V.) include both the intra-building and the inter-building variability. The comparison shows that the materials employed for the construction of the building specimen exhibit strengths comparable with those found in situ. Brick compressive strength, f_b , and masonry Young's modulus, E_{m1} , are the only two parameters that exceed the average (by 83% and 52%, respectively), but are still below the maximum values observed in the existing building stock [3].

3. Instrumentation

A total of 38 accelerometers, 21 wire potentiometers, and 37 linear potentiometers was installed on the building to capture its structural response during the dynamic tests. The majority of them was mounted in the shaking direction, while some were also oriented transversely or vertically. A set of 52 degrees of freedom was monitored by accelerometers installed on the walls, on the floor diaphragm, and on the roof (Fig. 9). Linear potentiometers (Fig. 10) recorded the longitudinal and transverse displacements of the floor diaphragm and of the top of the East and West walls with respect to the rigid steel frame. These sensors also monitored the relative displacements between the floor and the North and South walls and the relative displacements between the roof and the North and South gables. Wire potentiometers (Fig. 10) recorded the in-plane deformation of the floor diaphragm, the in-plane response of the East squat wall, and the out-of-plane displacement of the North and South façades.

A three-dimensional optical motion-capture system was also employed [19,20]. Passive spherical markers coated with a retro-reflective material were placed on the West, North, and South façades. Fixed cameras recorded in-plane and out-of-plane displacements of the monitored points during the earthquake simulations, allowing descriptions of local deformations and global displacements of the buildings.

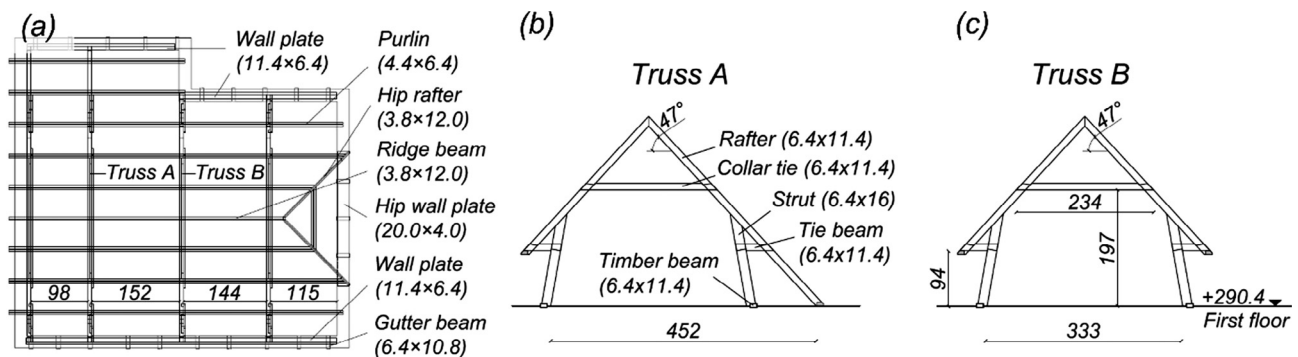


Fig. 6. Roof framing: (a) roof framing plan; (b) roof truss spanning between the East and the outermost West walls; (c) roof truss spanning between the East and the reentrant West walls (units of cm).

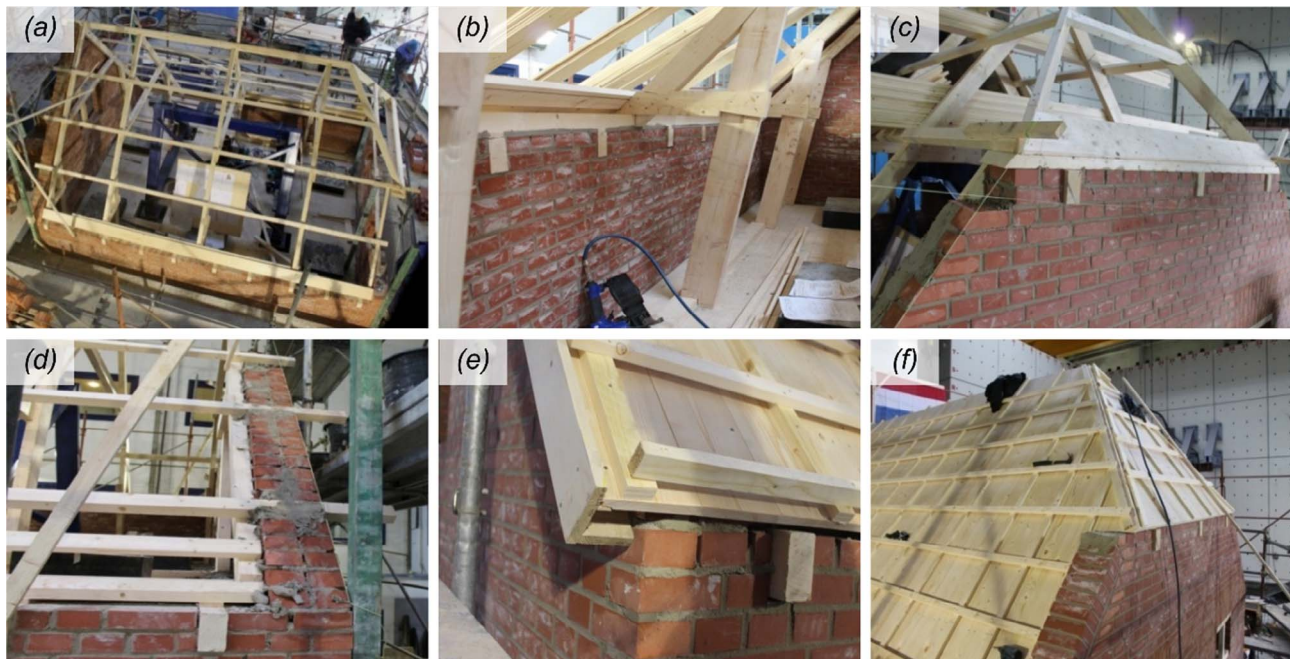
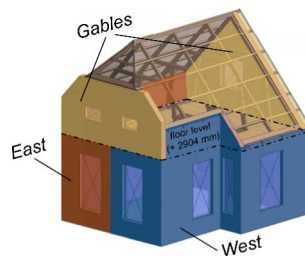


Fig. 7. Details of the roof: (a) roof framing; (b) roof support at East wall; (c) roof support at North gable; (d) purlins and truss at South gable; (e) planks blocking the purlins outside the South gable; (f) laths and counter battens mesh.

Table 1
Summary of structural and additional masses.

Component				Mass [t]
Masonry walls	East side	Below floor		8.75
		Above floor		2.21
	West side	Below floor		10.5
		Above floor		1.17
	Gables			6.29
Floor	Timber structure			0.47
	Additional floor mass			1.31
Roof	Timber structure			0.54
	Clay tiles			1.33
Total				32.6



All data, including both traditional and optical measurements, are available upon request on www.eucentre.it/nam-project. The reader is also referred to the companion data article [21], where a thorough description of the measurements obtained in the lab is provided.

4. Testing protocol

The specimen was subjected to incremental dynamic tests, applying a series of shake-table motions of increasing intensity to assess damage evolution, failure modes, and ultimate capacity of the building. Two accelerograms with different intensities that were characterized by smooth response spectra were proposed by Bommer et al., 2015 [22]: a first record, labelled SC1, with peak ground acceleration $PGA = 0.096$ g; and a second record, termed SC2, had $PGA = 0.155$ g. Fig. 11 shows the theoretical acceleration time-histories of the two input inputs and their elastic pseudo-acceleration response spectra at 5% viscous damping ratio.

These two records were then scaled in acceleration amplitude to obtain the desired incremental test protocol, consisting of 12 significant input ground motions. Table 3 illustrates the applied testing sequence specifying the input record; the amplitude scale factor; the nominal and recorded peak ground accelerations; the recorded peak ground velocities, PGV ; and the actual pseudo-spectral accelerations, S_a , and

displacements, S_d , at the undamaged fundamental period of the prototype for 5% damping ratio. After the strong-motion test SC2 - 400%, with recorded $PGA = 0.679$ g, the building was subjected to further testing at $PGA = 0.283$ g (SC2 - 200%-A), to simulate the effect of an aftershock on a damaged structure.

Other intensity measures were evaluated to characterize the input shake-table motions for possible correlations with the specimen performance. Among them, the conventionally defined cumulative absolute velocity, CAV , and Arias intensity, I_A , are listed in Table 3. The Arias intensity [23] can simultaneously reflect multiple attributes, such as the frequency content, the duration, and the amplitude of the ground motion; Travasarou et al. [24] have demonstrated that structural damage has a stronger correlation with I_A than that with PGA .

Extensive literature is available about the influence of ground motion duration on the structural demand [25]. The significant durations, D_{5-75} and D_{5-95} , were chosen as intensity measures, defined as the time intervals between the development of 5% and 75% of I_A , and between 5% and 95% of I_A , respectively. The unscaled record SC1 had $D_{5-75} = 0.39$ s and $D_{5-95} = 4.36$ s; the record SC2 had instead $D_{5-75} = 1.73$ s and $D_{5-95} = 7.09$ s. As each signal was linearly scaled in acceleration amplitude, its significant duration remained unchanged. This may appear counterintuitive, because higher intensity is usually associated with larger magnitude and hence longer duration. However,



Fig. 8. Mechanical characterization tests: (a) compression test on a clay brick; (b) three-points bending test on a clay brick; (c) three-points bending test on a mortar specimen; (d) compression test on a mortar specimen; (e) compression test on a masonry wallette; (f) four-points in-plane bending test; (g) four-points out-of-plane bending test; (h) bond wrench test; (i) shear test on a triplet.

Table 2

Masonry mechanical properties for the building prototype compared to in-situ tests on pre-1940 s clay-brick URM buildings in the Groningen region.

Material property [units]	Building prototype		In situ tests	
	Average	C.o.V.	Average	C.o.V.
Density of bricks, ρ_b [kg/m ³]	2101	0.02	–	–
Density of masonry, ρ [kg/m ³]	1984	0.01	–	–
Brick compressive strength, f_b [MPa]	46.8	0.11	25.6	0.23
Brick flexural strength, f_{bt} [MPa]	8.50	0.05	6.43	0.64
Mortar compressive strength, f_c [MPa]	4.12	0.24	–	–
Mortar flexural strength, f_t [MPa]	1.20	0.33	–	–
Masonry compressive strength, f_m [MPa]	9.23	0.07	8.91	0.52
Masonry Young's modulus in compression, E_{m1} [MPa]	8123	0.25	5346	0.60
Masonry flexural in-plane strength, f_{x3} [MPa]	0.44	0.19	0.61	0.45
Masonry flexural out-of-plane strength, f_{x2} [MPa]	0.64	0.15	0.83	0.47
Masonry flexural bond strength, f_w [MPa]	0.23	0.60	0.33	0.69
Masonry (bed-joint) cohesion, f_{v0} [MPa]	0.15	–	0.28	0.26
Masonry (bed-joint) shear friction coefficient, μ [–]	0.55	–	0.66	0.18

this effect can be partially compensated since larger PGA may also correspond to shorter epicentral distance and thus shorter duration. Moreover, in a probabilistic seismic hazard assessment, greater values of acceleration can be due to large positive residuals in the ground-motion prediction equation, and negative correlation has been found between the residuals of accelerations at medium-to-short frequencies and significant durations [26]. Strongly negative correlation has also

been observed between the residuals of PGA and D_{5-75} for the ground motions recorded at the Groningen field [27].

Before each test listed in Table 3, the building was subjected to low-amplitude random excitations covering a wide frequency band with consistent energy content. These additional tests allowed to assess the progressive effect of damage on the structural dynamic properties. In particular, the fundamental period of the undamaged structure was

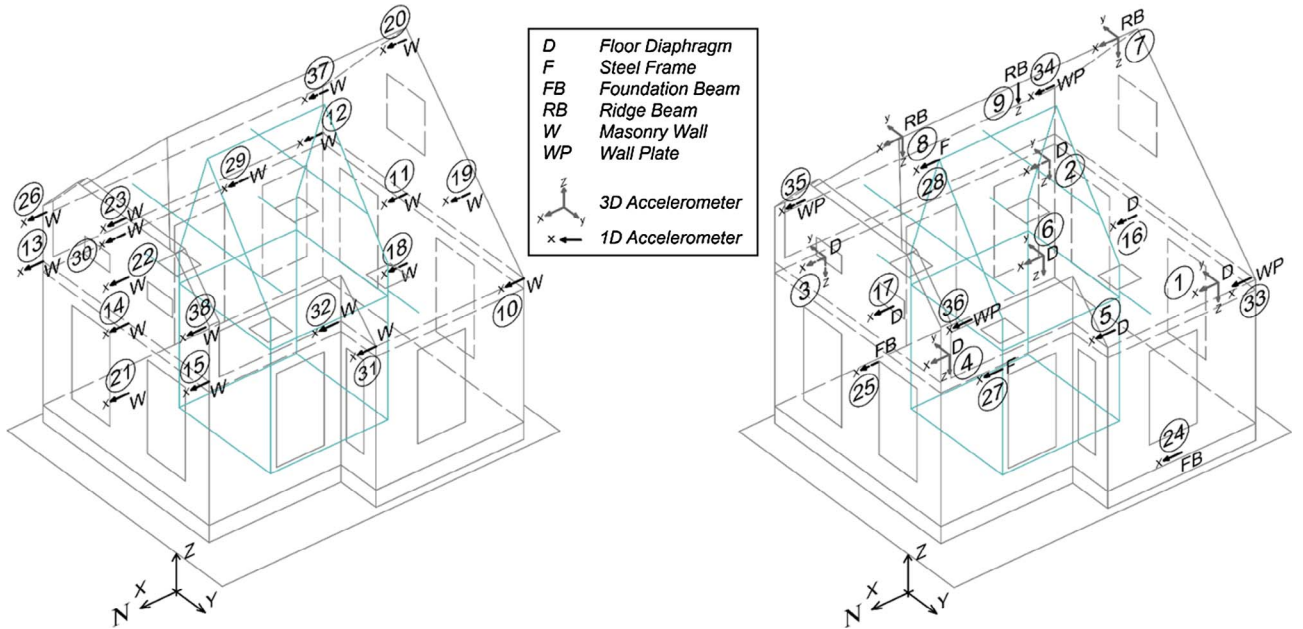


Fig. 9. Instrumentation plan: 3D and 1D accelerometers. Letters indicate the component on which each instrument is mounted. Rigid steel frame illustrated in light blue colour. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

$T_{1,und} = 0.10$ s, while at the end of the testing sequence it shifted to $T_{1,dam} = 0.22$ s. The average pseudo-spectral acceleration between $T_{1,und}$ and $T_{1,dam}$ was calculated according to Bianchini et al. [28], taking the geometric mean of the pseudo-acceleration spectrum at 5% viscous damping ratio:

$$\ln S_{a,avg} = \frac{1}{T_{1,dam} - T_{1,und}} \int_{T_{1,und}}^{T_{1,dam}} \ln S_a(T) dT. \quad (1)$$

While the geometric and arithmetic mean values are generally very similar (in this case, they differ by less than 0.7%), the former is less sensitive to extreme spectral ordinates [29]. Finally, a modified definition of the Housner intensity [30] was adopted. This intensity measure was redefined as the integral of the pseudo-velocity spectrum at 5% viscous damping between 0.1 s and 0.5 s, since the fundamental period of the examined structures is expected to be within this range [31]. This parameter has proven to be well correlated with the nonlinear displacement demand on short-period URM structures [32]:

$$mHI = \int_{0.1s}^{0.5s} PSV(T) dT. \quad (2)$$

5. Test results

The building did not suffer any visible damage up to the SC1 - 150% test ($PGA = 0.149$ g), began showing minor cracks under the SC2 - 150% motion ($PGA = 0.227$ g), and was considered at near-collapse state after the SC2 - 400% test ($PGA = 0.679$ g). Videos of the testing sequence are available on the EUCENTRE YouTube channel [33]. The following sections illustrate the performance of the specimen reporting qualitative damage observations, deformed shapes, and significant hysteretic response plots.

5.1. Shake-table performance

The performance of the shake-table test was deemed satisfactory in terms of response spectra of the actual table motion within the period range of the building specimen. Comparisons between the target response spectra and those obtained from the actual table motions, recorded at the specimen foundation level, demonstrate that the controller well reproduced the ground-motion for both low and high

intensity shakings (Fig. 12). A slight undershoot of the order of 10% is observed for spectral ordinates between 0.3 s and 0.7 s periods during application of the strong motion SC2 - 400%, when the structure exhibited significant inelastic response. A quantitative comparison between the actual and the target shaking intensity of SC2 - 400% shows that the actual intensity was almost equal to the expected one in terms of mHI (0.23 m instead of 0.25 m) and slightly larger in terms of CAV (7.37 m/s instead of 6.32 m/s). Discrepancies between the target and the recorded PGA in Table 3 are within the expected range of current abilities of shake-table controllers to reproduce a given ground motion when shaking a nonlinear structure.

5.2. Damage evolution

At the end of every shaking test, structural damage was surveyed in detail and cracks were accurately mapped. Fig. 13 shows the evolution of the overall damage pattern. Cracks marked in red and blue were observed at the end of the annotated test run on the internal and external side of the walls, respectively. Cracks shown in black had already been detected after previous shaking runs.

During testing under SC1 input motions, scaled from 25% to 150% (recorded PGA from 0.026 g to 0.149 g), the building did not experience any visible damage. Similarly, the SC2 signal scaled at 50% and 100% (PGA from 0.080 g to 0.140 g) did not induce any detectable crack.

Minor damage became visible on the North clipped gable during testing under SC2 - 150% ($PGA = 0.227$ g): horizontal hairline cracks were found just above the openings, extending throughout the façade length. Hairline cracks were also observed at the top of the piers of the protruding West wall portion, and between the roof purlins and the supporting masonry of the North gable. Damage did not evolve significantly after testing under SC2 - 200% ($PGA = 0.293$ g). The SC2 - 250% test ($PGA = 0.392$ g) resulted in horizontal hairline cracks forming on the South façade, a few centimetres above the floor level. The crack pattern developed up to this stage was compatible with incipient activation of out-of-plane overturning mechanisms of both gables. A diagonal hairline crack formed on the West wall above the corner of the northern window, extending towards the North façade, due to interaction between in-plane and out-of-plane responses of the intersecting walls. Hairline cracks developed also at the bottom of the wider piers of the West façade, indicative of their rocking response. No

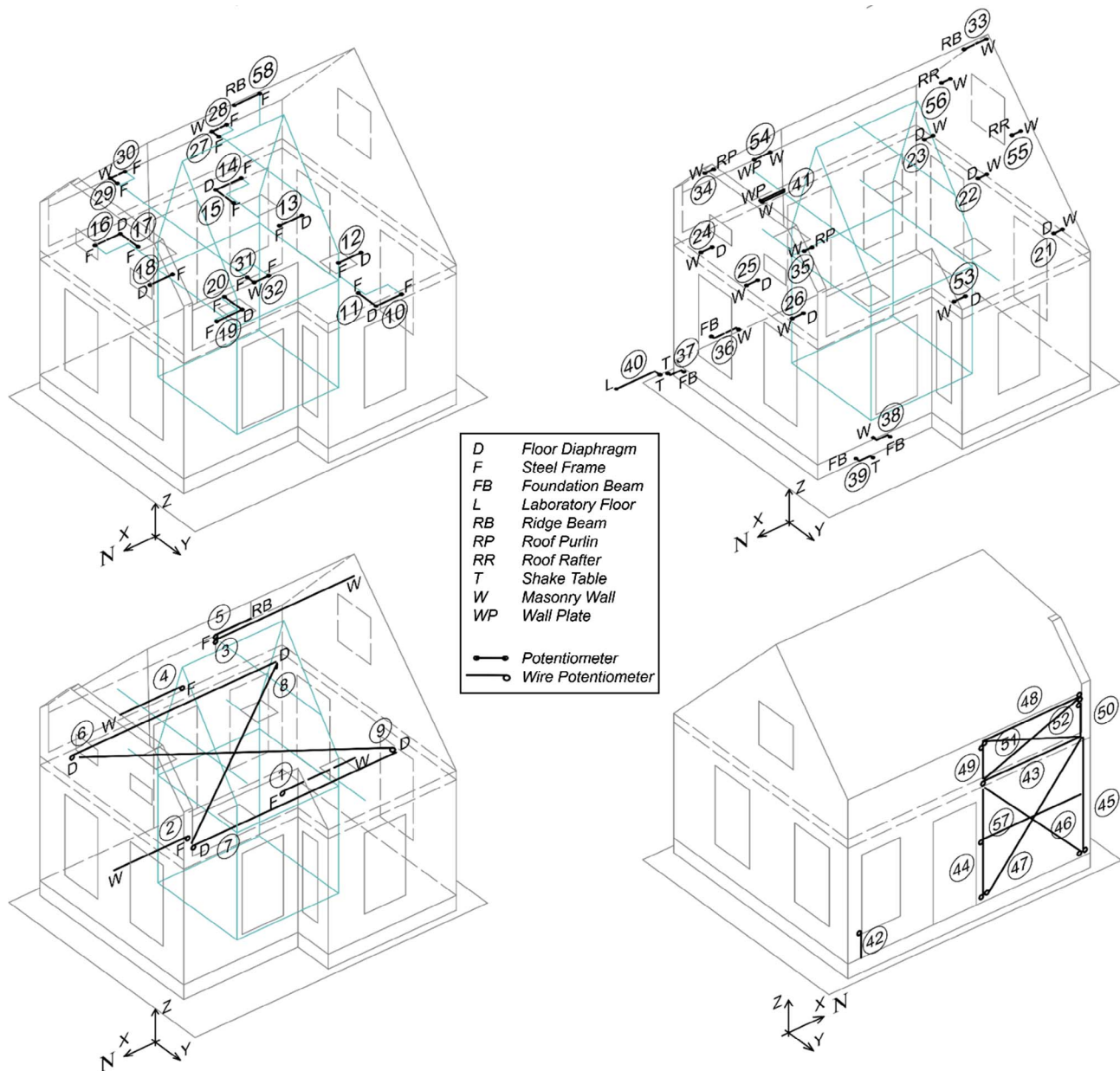


Fig. 10. Instrumentation plan: linear and wire potentiometers. Letters indicate the component on which each instrument is mounted. Rigid steel frame illustrated in light blue colour. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

damage was detected on the East wall until this intensity level. Overall, the structure suffered only slight damage and was deemed fully operational with just minor repairs.

During shaking under SC2 - 300% ($PGA = 0.500\text{ g}$) a global response of the structure was triggered, as evidenced by the formation of new cracks and the propagation of pre-existing ones. Out-of-plane mechanisms developed on both gables. A 45° -diagonal crack was clearly observed in the centre of the South gable: this was attributed to unequal out-of-plane displacements at the intersections with the East and West walls, or to the restraining effect of the steel anchor Y1 at the floor level. Part of the East façade participated in the out-of-plane response of the South gable due to good interlocking between the intersecting walls, as testified by a vertical crack with residual width of 0.8 mm above the southern window. A portion of the West façade was involved in the out-of-plane mechanism of the North wall: the pre-existing diagonal crack reached approximately 1-mm permanent opening. In-plane mechanisms developed in the West longitudinal piers with prevailing flexural-rocking behaviour, as suggested by the propagation

of hairline cracks at their top and bottom. The building was brought to a moderate structural damage condition, requiring extensive repairs and possible disruption of its functionality.

After the SC2 - 400% test ($PGA = 0.679\text{ g}$) widespread damage was observed throughout the building, which was deemed to have reached near-collapse conditions. An out-of-plane rigid-body mechanism involved the upper portion of the North gable: damage included mortar joint sliding, with a maximum residual displacement of the order of 20 mm, and block de-cohesion along a pre-existing horizontal crack just above the openings (Fig. 14a and b). An out-of-plane overturning mechanism fully activated also on the South gable. X-shaped crack patterns developed close to midspan of both gables at the floor level (Fig. 14c) with permanent opening up to 2 mm, mainly attributed to the restraining effect of the steel anchors Y1 which also caused local brick cracking (Fig. 14d). Participation of parts of the East and West walls in the out-of-plane response of the North and South façades became evident: stair-stepped and vertical cracks propagated in the upper areas of the longitudinal walls at the intersection with the transverse ones

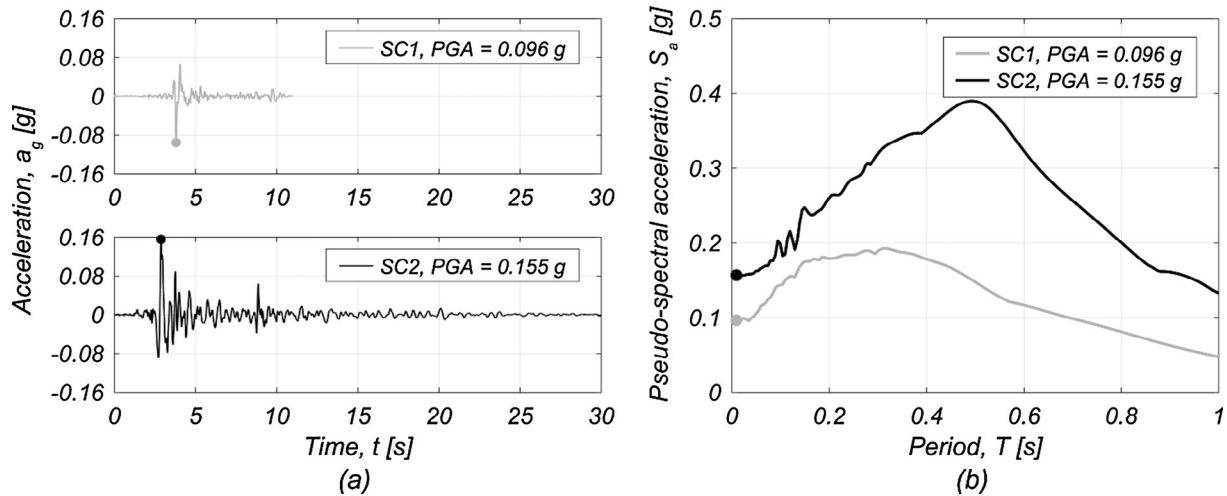


Fig. 11. SC1 and SC2 signals: (a) acceleration time histories; (b) pseudo-acceleration elastic response spectra for 5% viscous damping ratio.

(Fig. 14e and f), with permanent width ranging from 1.5 to 8 mm.

All West piers exhibited pure flexural-rocking failure mechanisms: pre-existing cracks propagated at their top and bottom, reaching residual width between 1.5 and 10 mm (Fig. 14g and h); one of the flat-arch lintels was also severely damaged (Fig. 14i). Visible cracks formed on the East squat pier only at this final stage of the test, associated with a hybrid sliding/rocking response: permanent sliding of about 0.2 mm was noticed all along its base and a diagonal crack with 0.2-mm residual opening was detected close to the North-East corner (Fig. 14j). The East slender piers exhibited cracks characteristic of flexural-rocking behaviour, with residual opening limited to 1 mm. The final crack pattern of Fig. 13 shows the contribution of portions of the North and South façades as flanges for the in-plane response of the longitudinal corner piers.

Despite weak connections between roof purlins and gables, and between wall plates and longitudinal walls, sliding or significant differential displacements were not observed nor recorded by any instruments across these interfaces. Failures of some nailed connections between the roof trusses and the wall plates allowed relative translation between the roof diaphragm and the longitudinal walls top (Fig. 14k). Some damage due to pounding between the roof sheathing and the North gable was also noticed (Fig. 14l).

The building withstood an aftershock equal to SC2 - 200% with minor widening of pre-existing cracks.

5.3. Identification of damage limit states

This section proposes qualitative definitions of damage states (DS) for the clay URM building specimen with reference to the observations

during the testing sequence. It also identifies thresholds between the damage states, termed damage limits (DL), and relates them to quantitative engineering demand parameters.

Five damage states were considered: DS1, no structural damage; DS2, minor structural damage; DS3, moderate structural damage; DS4, heavy structural damage; and DS5, very heavy structural damage with partial or total collapse [7,34–36]. Four damage limits were consequently defined as follows: DL1, the limit condition at which no structural damage was visible; DL2, the limit condition at which only minor structural damage was detected; DL3, the limit condition at which only moderate structural damage was reported; DL4, the limit condition at which heavy structural damage was detected, before entering the near-collapse conditions. In other words, DL(*i*) constitutes the threshold between DS(*i*) and DS(*i* + 1).

Due to the different geometries of the longitudinal East and West walls, combined with the flexibility of roof and floor diaphragms, damage limits were first identified locally considering three separate subsystems: East wall, West wall, and gable-roof assembly. For a given subsystem, each DL was associated with an earthquake input: specifically, DL(*i*) was associated with the last run that caused damage classified as DS(*i*). The maximum interstorey drift ratio induced on the subsystem by this run was taken as the reference engineering demand parameter corresponding to DL(*i*). Interstorey drift ratios $\theta_{1,E}$ and $\theta_{1,W}$ were defined individually for the East and West walls, respectively, at the first floor level:

$$\theta_{1,E} = \frac{\Delta_{1,E}}{h_1} \quad \text{and} \quad \theta_{1,W} = \frac{\Delta_{1,W}}{h_1}, \quad (3)$$

where $h_1 = 2904$ mm is the first-floor height above the foundation, and

Table 3
Summary of the testing sequence.

Test input	Scale factor	Nom. PGA [g]	Rec. PGA [g]	PGV [m/s]	$S_a(T_{1,und})$ [g]	$S_d(T_{1,und})$ [mm]	CAV [m/s]	I_A [mm/s]	$S_{a,avg}$ [g]	mHI [mm]
SC1	25%	0.024	0.026	0.013	0.04	0.10	0.22	1.5	0.05	8.4
SC1	50%	0.048	0.050	0.027	0.07	0.18	0.36	5.3	0.09	17
SC1	100%	0.096	0.098	0.055	0.15	0.37	0.54	20.0	0.17	34.2
SC1	150%	0.144	0.149	0.082	0.22	0.55	0.82	45.7	0.25	51.7
SC2	50%	0.077	0.080	0.063	0.10	0.25	0.98	29.7	0.12	32.1
SC2	100%	0.155	0.140	0.110	0.17	0.42	1.85	100.3	0.21	56.6
SC2	150%	0.232	0.227	0.175	0.28	0.70	3.09	265.4	0.33	90.3
SC2	200%	0.310	0.293	0.228	0.38	0.94	4.13	453.2	0.43	116.3
SC2	250%	0.387	0.392	0.297	0.50	1.23	5.43	775.5	0.57	151.1
SC2	300%	0.465	0.500	0.346	0.62	1.54	5.80	975.1	0.7	177
SC2	400%	0.620	0.679	0.444	0.87	2.16	7.37	1601.7	0.94	227.1
SC2	200%-A	0.310	0.283	0.222	0.38	0.94	4.18	432.5	0.43	122.1

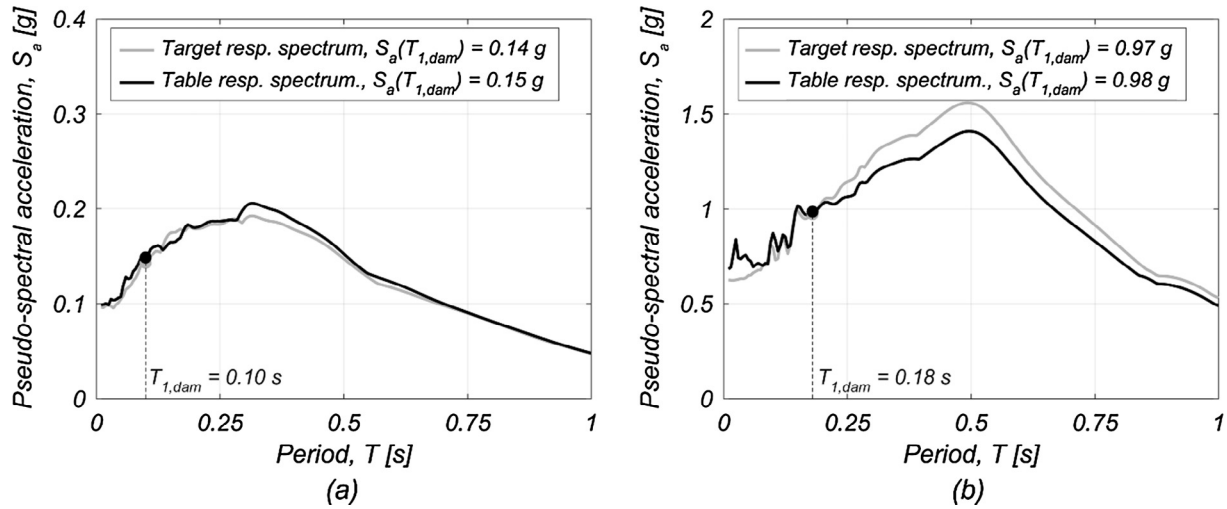


Fig. 12. Comparison between elastic 5%-damped pseudo-acceleration response spectra of the actual input records and target spectra: (a) test SC1 - 100% (PGA = 0.098 g); and (b) test SC2 - 400% (PGA = 0.679 g).

$\Delta_{1,E}$ and $\Delta_{1,W}$ are the average displacements measured on top of the two walls with respect to the foundation. The average first-floor drift ratio, $\theta_{1,AVG}$, was taken as the mean between East and West walls:

$$\theta_{1,AVG} = \frac{\theta_{1,E} + \theta_{1,W}}{2}. \quad (4)$$

The roof-storey drift ratio was defined as the ratio of the relative displacement between ridge and first floor, to the ridge height above the first floor, $h_R = 3330$ mm:

$$\theta_R = \frac{\Delta_R - \Delta_{1,AVG}}{h_R}. \quad (5)$$

where Δ_R and $\Delta_{1,AVG}$ represent the roof ridge and average first-floor displacements with respect to the foundation. Although the identification of a collapse prevention limit (DL4) is not straightforward, due to test termination before reaching partial or total collapse (DS5), the permanent opening of wide cracks and the dislocation of masonry portions were considered limit conditions for DS4.

Table 4 lists the test runs when each subsystem reached the damage thresholds. The East wall exhibited a combined rocking-sliding damage mechanism that was activated only during the final test, SC2 - 400% (PGA = 0.679 g). This longitudinal wall underwent low deformations, due to the high in-plane strength and stiffness of the squat pier compared to the more flexible slender piers of the West façade; the deformation capacity of the East wall was not fully exploited and therefore it developed only moderate damage, reaching the DL3 threshold at the end of the sequence. On the other hand, the West façade underwent severe damage that impaired substantially its lateral load-carrying capacity, to the point that DL4 was associated with its final conditions. Also the gable-roof subsystem reached DL4, with development of gable overturning mechanisms, permanent dislocation of the North gable, and failure of several nailed connections of the timber roof.

Once the local damage limits were established for each subsystem, overall damage limits were identified for the entire building (Table 4). Each overall damage limit DL(i) was associated with the first test run that caused any subsystem to reach that threshold locally, as reported on Fig. 13. This choice is justified by the low redundancy of the structural system: for instance, if either the East or the West wall collapsed, the simply supported floor beams and roof trusses would also collapse. In case of structural assemblies with higher static indeterminacy, different relationships between local and global damage limits could be more appropriate.

The pier deformations corresponding to the local damage limits for the West façade can be compared with the deformations associated with

the element damage limits from quasi-static cyclic shear-compression tests on individual piers. In order to perform this comparisons, the element drift ratios were approximately estimated for the piers of the West façade, by multiplying the interstorey drift angles times the ratio of the storey height (2.90 m) to the pier effective height (nearly equal to the 1.86-m window height).

A large database has been recently compiled [37] which summarizes the results of 188 in-plane quasi-static cyclic shear-compression tests on various masonry pier typologies. Despite the list includes 30 tests on solid clay brick piers, only three were tested up to failure and had characteristics comparable with the piers of the building specimen: thickness equal or less than 250 mm, aspect ratio equal or greater than 2 and vertical compressive stress-to-strength ratio equal or less than 10%. These three pier specimens were the ones constructed in parallel with the full building specimen [6], using the Dutch cross bond and providing an aspect ratio similar to those of the West façade piers. The database provides information about pier drift-ratios associated with first visible cracking, maximum lateral-force resistance, ultimate conditions, and yielding in an elastoplastic idealization of the backbone curve.

The element drift ratios associated with DL2 and DL3 for the West façade (0.095% and 0.75%) are slightly larger than the average values corresponding to idealized yielding and maximum lateral resistance of the three piers of the database (0.068% and 0.53%), respectively. The West façade was able to withstand large displacements, maintaining pier integrity during repeated rocking cycles and exhibiting only early signs of toe-crushing damage. In fact, the façade maximum element drift demand (2.5%) exceeded the average ultimate capacity of the three statically tested piers (1.7%); however, it was lower than the ultimate capacity (3.0%) of the pier with the lowest overburden load, which was closer to the static conditions of the building piers and exhibited a pure flexural failure mode. The drift ratios associated with damage limits in the shake-table test and in the cyclic shear-compression tests may be differently affected by cumulative damage: this may result in a more conservative estimate of the lateral displacement capacity from shear-compression tests with several cyclic repetitions.

5.4. Deformed shapes

The damage mechanisms experienced by the building prototype at different stages of the dynamic test can be visualized from the deformed shapes presented on Fig. 15.

From the plan views of Fig. 15a one can observe differential x-displacements between the two points of each longitudinal East or West



Fig. 13. Evolution of the specimen crack pattern. Cracks marked in red were observed from inside, in blue from outside. Cracks marked in black were already detected in previous test runs. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

façades: this was due to crack opening, which caused elongation of the walls. The West side, consisting of slender piers with rocking behaviour, exhibited larger displacements than the East façade, which included a stiffer and stronger squat pier. Consequently, the floor diaphragm underwent significant shear deformation, but its flexibility prevented overall torsional response of the building (Fig. 15a). For instance, at the instant of peak shear deformation of the floor diaphragm ($\gamma_{f,max} = 0.98\%$) during the SC2 - 400% test, the longitudinal displacement at the first-floor level was 47.4 mm on the West façade, about 5.5 times the displacement of 8.6 mm recorded on the East wall.

Displacement profiles at midspan of the transverse walls (Fig. 15b)

show initial interstorey displacement concentrations between the roof ridge and the first floor, dominated by out-of-plane response of the gable-roof system. At the instant of maximum ridge displacement ($\Delta_{R,max} = 24.3$ mm) during the test SC2 - 250%, a roof drift ratio of 0.64% was recorded, which is about 6.5 times the first-storey drift ratio of 0.10% recorded at the South wall. During the final runs, instead, the displacement profile became more uniform over the building height, due to activation of rocking mechanisms on the West façade. The maximum ridge displacement ($\Delta_{R,max} = 131.5$ mm) recorded during test SC2 - 400% corresponded to a roof drift ratio of 2.9%, only about 2.5 times the first-storey drift ratio of 1.2% measured at the South wall.

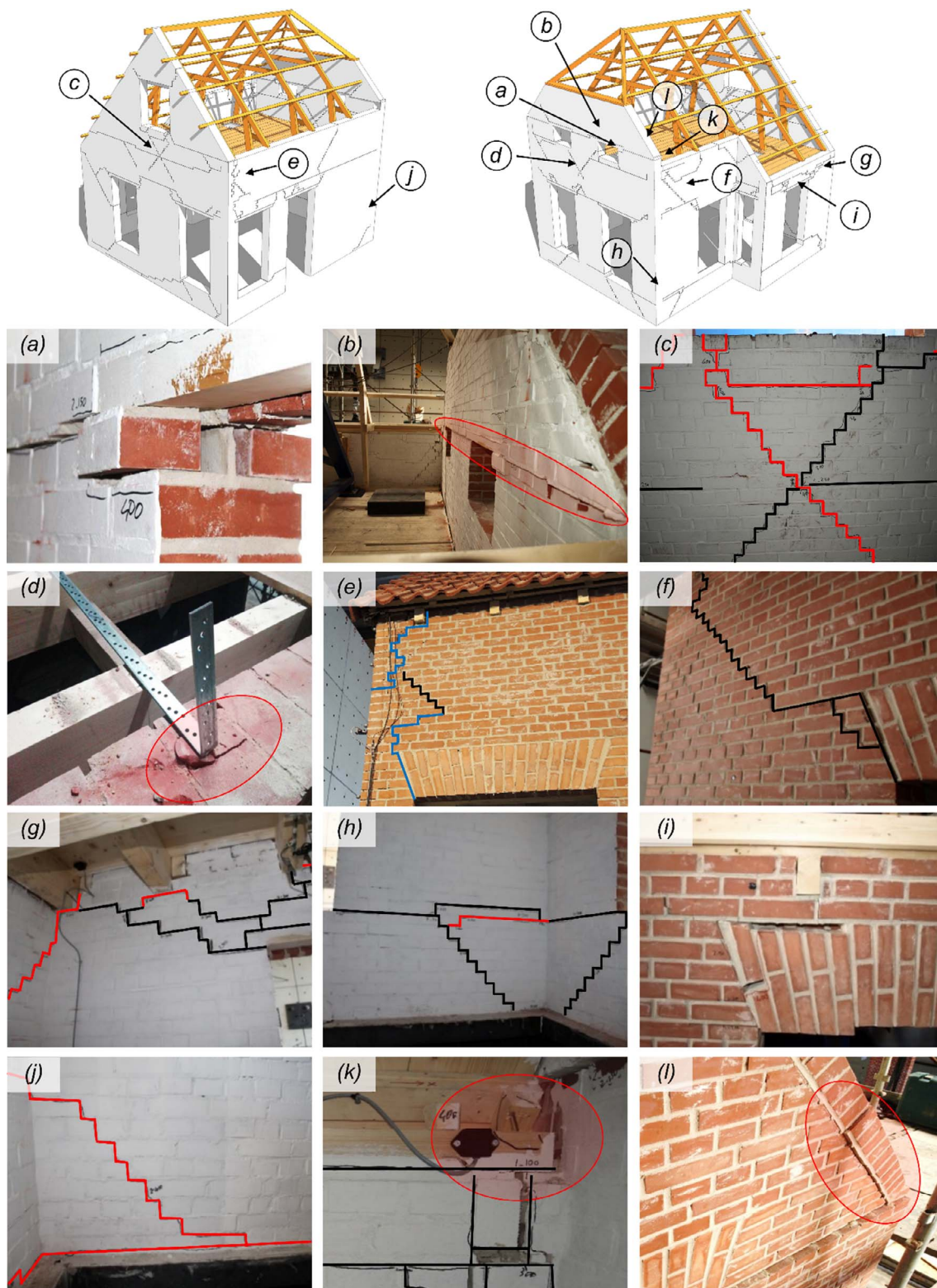


Fig. 14. Observed damage at the end of test SC2 - 400%: (a) block de-cohesion on the North gable; (b) mortar-joint sliding above the openings of the North gable; (c) X-shaped cracks on the South gable; (d) brick cracking at the location of the steel anchor Y1; (e) cracks on the East façade due to the out-of-plane mechanism of the South gable; (f) cracks on the West façade due to the out-of-plane mechanism of the North gable; (g) flexural cracks at the top of a West pier; (h) flexural cracks at the base of a West pier; (i) near-collapse state of a flat-arch; (j) flexural and sliding cracks at the base of the East squat pier; (k) failure of a nailed connection of the timber roof; (l) damage due to pounding of the roof girders on the North gable.

5.5. Hysteretic response

Fig. 16 shows the hysteretic response of the specimen in terms of normalized base-shear versus interstorey drift ratio during the last three

tests. Hysteretic plots are provided for each one of the three considered subsystems, namely, East wall, West wall, gables-roof assembly, and for the overall structure. White dots represent the points of maximum base-shear demand, while black dots correspond to the points of maximum

Table 4

Summary table of damage limit states for the building specimen. Colors identify damage limits consistently with Fig. 13.

	DL1	DL2	DL3	DL4
West wall	SC2-200% ($PGA = 0.29$ g) $\theta_{1,W,DL1} = 0.016\%$ No visible damage other than hairline cracks on top of the piers of the outermost portion.	SC2-250% ($PGA = 0.39$ g) $\theta_{1,W,DL2} = 0.061\%$ Hairline flexural/rocking cracks at top and bottom of the wider piers. Hairline diagonal crack due to interaction with the North gable.	SC2-300% ($PGA = 0.50$ g) $\theta_{1,W,DL3} = 0.48\%$ Hairline rocking cracks on all piers. Diagonal crack due to interaction with the North gable, with residual width of 1 mm.	SC2-400% ($PGA = 0.68$ g) $\theta_{1,W,DL4} = 1.6\%$ Rocking cracks on all piers with residual width up to 10 mm; brick de-cohesion in one of the flat-arch lintels. Diagonal crack due to interaction with the North gable, with residual width of 4 mm.
East wall	SC2-250% ($PGA = 0.39$ g) $\theta_{1,E,DL1} = 0.016\%$ No visible damage.	SC2-300% ($PGA = 0.50$ g) $\theta_{1,E,DL2} = 0.039\%$ No cracks due to in-plane pier response. Vertical crack due to interaction with the South gable, with residual width of 0.8 mm.	SC2-400% ($PGA = 0.68$ g) $\theta_{1,E,DL3} = 0.31\%$ Rocking cracks on the slender piers, with residual width of 1 mm; rocking and sliding cracks on the squat pier, with residual width of 0.2 mm. Vertical crack due to interaction with the South gable, with residual width of 8 mm.	Not reached.
Gable-roof	SC2-100% ($PGA = 0.14$ g) $\theta_{R,DL1} = 0.063\%$ No visible damage.	SC2-250% ($PGA = 0.39$ g) $\theta_{R,DL2} = 0.70\%$ Hairline horizontal cracks on both gables, due to incipient out-of-plane overturning mechanisms.	SC2-300% ($PGA = 0.50$ g) $\theta_{R,DL3} = 1.2\%$ Activation of out-of-plane mechanisms on both gables.	SC2-400% ($PGA = 0.68$ g) $\theta_{R,DL4} = 3.3\%$ Out-of-plane overturning mechanisms fully activated on both gables; sliding of the upper portion of the North gable, with residual of 20 mm. Damage to the roof nailed connections.
Overall	Global DL1: Maximum demand with no evident structural damage. Governed by the gable-roof assembly . SC2-100% ($PGA = 0.14$ g) $\theta_{1,AVG,DL1} = 0.005\%$	Global DL2: Maximum demand with only minor structural damage. Governed by West wall and gable-roof assembly . SC2-250% ($PGA = 0.39$ g) $\theta_{1,AVG,DL2} = 0.038\%$	Global DL3: Maximum demand with only moderate structural damage. Governed by West wall and gable-roof assembly . SC2-300% ($PGA = 0.50$ g) $\theta_{1,AVG,DL3} = 0.25\%$	Global DL4: Maximum demand with heavy structural damage before developing near-collapse conditions. Governed by West wall and gable-roof assembly . SC2-400% ($PGA = 0.68$ g) $\theta_{1,AVG,DL4} = 0.94\%$

drift-ratio demand. Northward displacements and forces are positive.

The base-shear coefficient, BSC , is defined as the base shear normalized by the weight of the mass that contributes to the same force. The base shear was computed as the sum of the products of each accelerometer measurement times the tributary mass, lumped at the instrument location. Considering the overall building and its East and West subsystems, these coefficients can be expressed as:

$$BSC_{TOT} = \frac{\sum a_i \cdot m_i}{g \cdot m_{TOT}}, \quad BSC_E = \frac{\sum a_{i,E} \cdot m_{i,E}}{g \cdot m_E}, \quad BSC_W = \frac{\sum a_{i,W} \cdot m_{i,W}}{g \cdot m_W}, \quad (6)$$

where a_i represents the acceleration recorded by accelerometer i , and m_i is the tributary mass associated with the instrument. Subscripts TOT , E and W identify quantities related to overall structure or its subsystems. The mass distribution illustrated in Table 1 was used to determine the total masses: $m_{TOT} = \sum m_i = 32.6$ t, $m_E = \sum m_{i,E} = 15.9$ t, and $m_W = \sum m_{i,W} = 16.7$ t. For the calculation of the subsystem shear forces, half of the inertia of transverse walls, floor, and roof was allocated to each of them.

The base-shear determined with Eq. (6) includes the inertia of the lower half of the first storey walls, which accelerate together with the shake-table. This mass is about 8.7 t and represents 27% of the total for

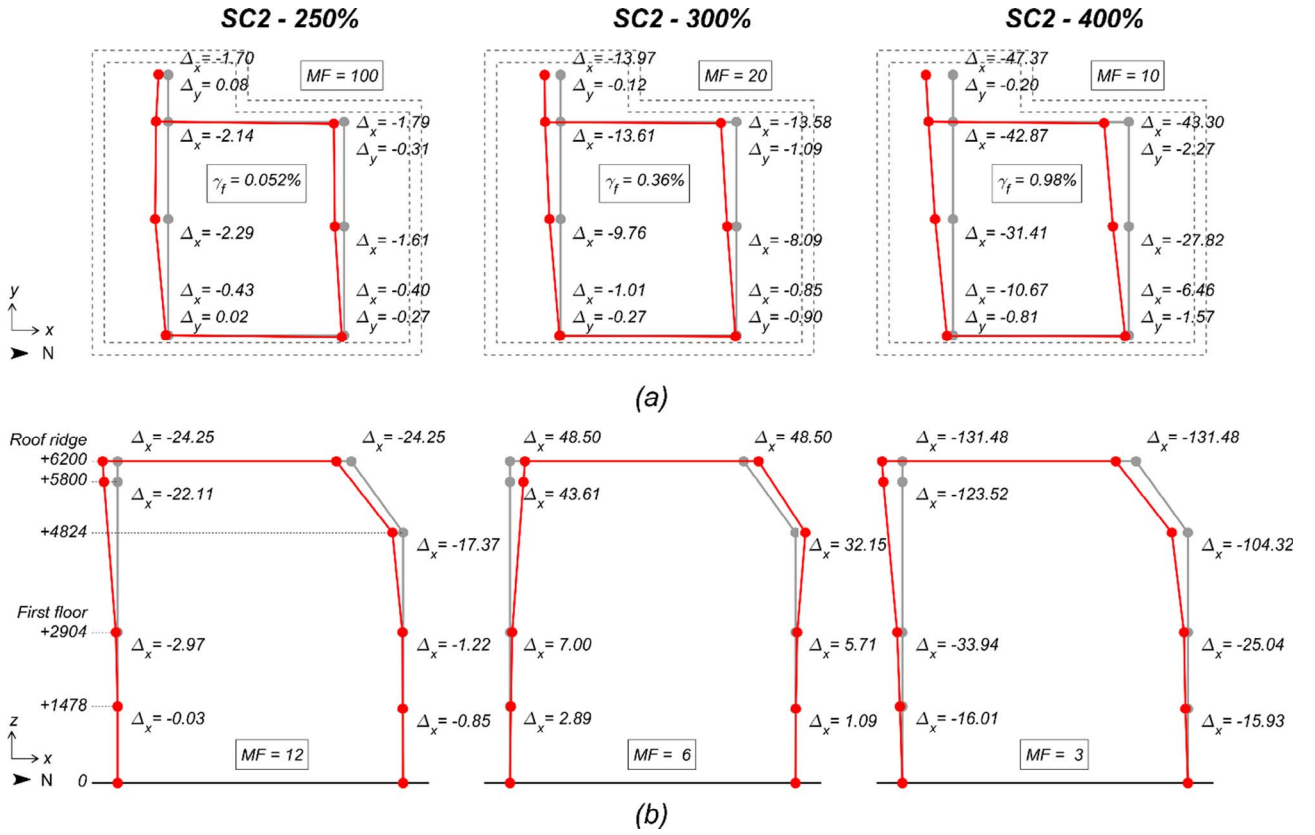


Fig. 15. Deformed shapes: (a) plan view of the first-floor displacements, at the instants of maximum diaphragm shear deformations; (b) displacement profiles at midspan of the transverse walls, at the instants of maximum roof ridge displacements. MF is the displacement magnification factor on the figure. Units of mm unless otherwise specified.

the building prototype. When using equivalent frame or other numerical models, the mass of lower half of the first storey walls is often assumed concentrated at the base and is supposed to move with the ground, without contributing to the seismic response of the structure. Therefore, base-shear coefficients BSC^0 can be determined excluding the contribution of the lower parts of the masonry walls:

$$BSC_{TOT}^0 = \frac{\sum a_i \cdot m_i^0}{g \cdot m_{TOT}^0}, \quad BSC_E^0 = \frac{\sum a_{i,E} \cdot m_{i,E}^0}{g \cdot m_E^0}, \quad BSC_W^0 = \frac{\sum a_{i,W} \cdot m_{i,W}^0}{g \cdot m_W^0}, \quad (7)$$

where m_i^0 is the tributary mass associated with each instrument excluding the portions accelerated with the table, $m_{TOT}^0 = \Sigma m_i^0 = 23.9$ t, $m_E^0 = \Sigma m_{i,E}^0 = 11.9$ t, and $m_W^0 = \Sigma m_{i,W}^0 = 12.0$ t. A comparison of the backbone curves obtained with the two base-shear definitions is provided in Fig. 17, considering the response in the negative direction.

Nonlinear overall response was initially observed in the SC2 - 250% test ($PGA = 0.392$ g) and became rapidly pronounced during the two following tests; it was associated with the formation of flexural-rocking cracks in West piers and out-of-plane behaviour of the gables. During test SC2 - 400% with $PGA = 0.679$ g, the maximum recorded average first-floor drift ratio was 0.94%, while the maximum attained overall base-shear coefficients were $BSC_{TOT,max}^0 = 0.54$ and $BSC_{TOT,max}^0 = 0.50$. The difference obtained from the two definitions of the maximum base-shear coefficient was less than 10% for the overall structure and for the East and West subsystems.

The performance of the structure was influenced by the large in-plane diaphragm flexibility, which provided very little coupling between the longitudinal walls and allowed them to displace almost independently. The almost bilinear nonlinear behaviour exhibited by the West side was indicative of pier rocking, while the hysteretic response of the East façade was predominantly linear, as most of the East wall was still undamaged at the beginning of the last run. A rapid increase in

nonlinear response and damage concentration was observed in the West façade, due to rocking of its slender piers: during the test SC2 - 250%, despite only light cracking at the base of the West piers, the first-storey drift ratio of 0.061% on the West façade was already almost 4 times the demand of 0.016% on the East wall. During test SC2 - 400%, rocking response of the slender piers induced significantly larger interstorey drifts on the West wall compared to the East wall: 1.6% compared to 0.31%. However, rocking behaviour of the West piers resulted in smaller residual drift ratios as opposed to the hybrid mechanism of the East squat pier: 0.02% as opposed to 0.03%, as illustrated on Fig. 18.

The response of the roof-gable assembly was analysed in terms of roof-storey drift ratio and roof-shear coefficient. Two definitions were adopted for the latter, as for the base shear. In one case, RSC was taken as the ratio between the storey shear at the roof base and the weight of the mass above the first-floor that contributes to that force. Following the second approach, only the masses located above half the roof storey height and the corresponding weights and lateral inertia were considered to calculate RSC^0 . In symbols:

$$RSC = \frac{\sum a_i \cdot m_{i,R}}{g \cdot m_R} \quad \text{and} \quad RSC^0 = \frac{\sum a_i \cdot m_{i,R}^0}{g \cdot m_R^0}. \quad (8)$$

where a_i and $m_{i,R}$ are the acceleration time-series and the tributary mass of accelerometer i , mounted on the roof-gable assembly, $m_R = \Sigma m_{i,R} = 11.54$ t (see Table 1), and $m_R^0 = \Sigma m_{i,R}^0 = 2.2$ t. The hysteretic response obtained with the first roof-shear definition is shown on Fig. 16. A comparison of the backbone curves obtained with both definitions is provided in Fig. 17.

Due to the composite nature of the roof-gable assembly structure, isolating the response of the timber diaphragm from that of the masonry gables was not possible. The gable-roof assembly exhibited almost linear elastic behaviour up to drifts of nearly 0.2%. The initial stiffness, calculated with the first roof-shear definition, was

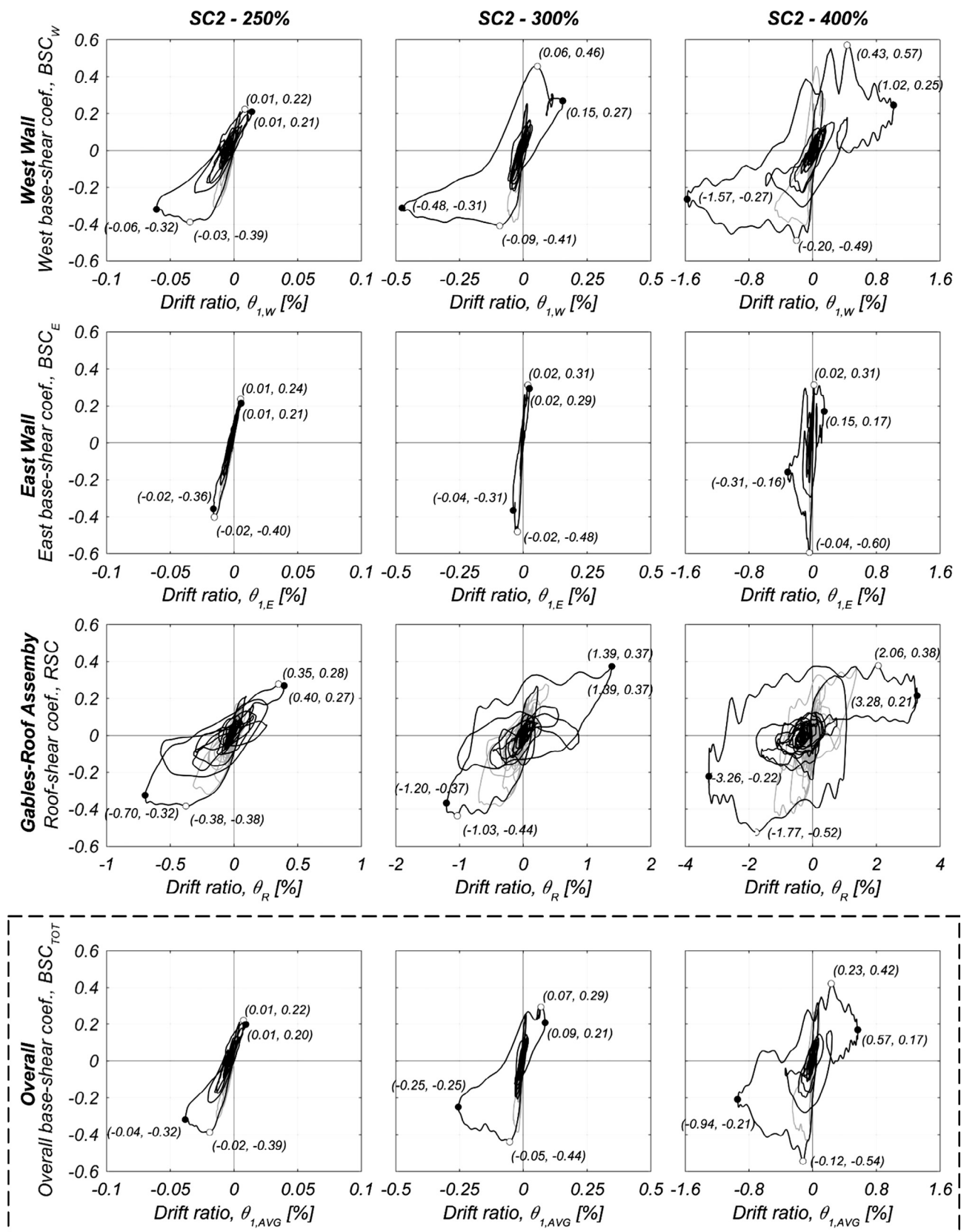


Fig. 16. Specimen hysteretic responses.

approximately 5.4 kN/mm. Beyond this response level the roof-gable assembly entered a nonlinear phase with a reduced stiffness of 0.45 kN/mm, less than 10% of the initial one. The wide hysteresis loops demonstrate that the roof timber diaphragm is capable of dissipating

considerable amounts of energy.

The peak roof-storey drift ratio was $\theta_{R,max} = 3.3\%$. The maximum roof shear coefficients were $RSC_{max} = 0.52$ and $RSC^0_{max} = 1.3$. For the roof-gable system the difference between the two approaches was

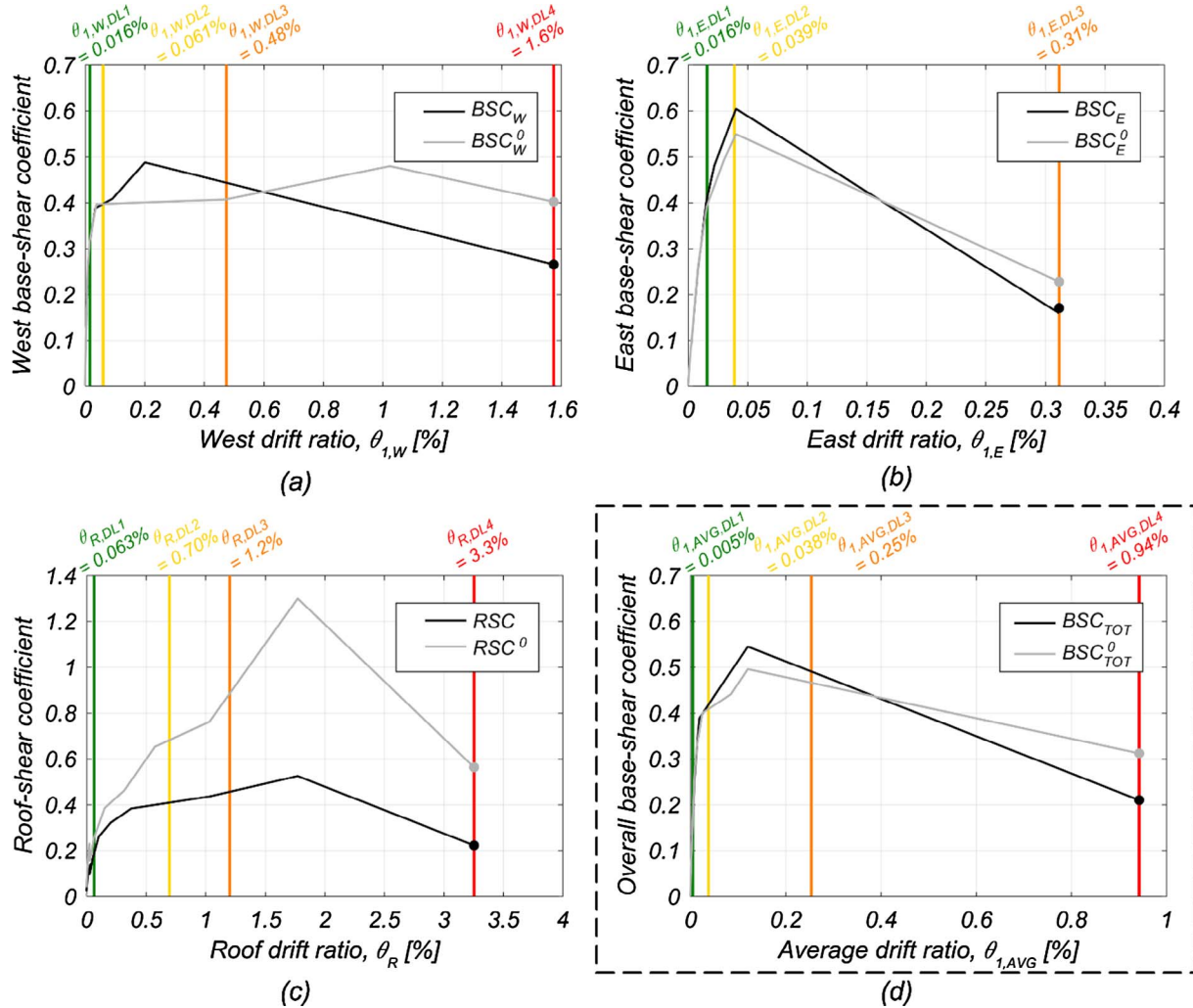


Fig. 17. Backbone curves and damage limit states: (a) West wall; (b) East wall; (c) gables-roof assembly; and (d) overall building response.

significant, with RSC_{max}^0 equal to 2.5 times RSC_{max} . The reason of this discrepancy can be found in the contribution of the upper part of the reentrant West wall (above the floor level) to RSC. In fact, the response of the roof-gable assembly and that of the West wall is not in phase, and the roof ridge tends to magnify the ground acceleration more than the longitudinal walls at the floor level (Fig. 18). Consequently, the reduction in weight from RSC to RSC^0 is larger than the reduction in lateral inertia force, resulting in larger RSC^0 .

The damage limits DL1 through DL4, defined in Section 5.3 for each subsystem and for the entire building, are associated with points of the force-displacement backbone curves on Fig. 17.

6. Summary

An overall summary of response of the tested structure is shown in Fig. 18 where peaks lateral displacements and peak and residual drift ratios are plotted for all test runs. The variation of peak acceleration-amplification factors and fundamental periods is also shown.

The acceleration amplification factors, AMP, is defined as the ratios of peak acceleration response recorded at various locations the peak ground (or table) acceleration. Fig. 18 illustrates the maximum AMP recorded at four locations: on the East and West masonry walls at the first-floor level, on the floor timber diaphragm, and on the roof ridge. The East-wall maximum AMP fluctuated around 1.1 throughout the experiment. The one of the West wall also varied slightly around 1.1 until test SC2 - 200%, but cracking of the West piers during run SC2 -

300% corresponded to a sudden increase to 2.0. However, the West wall AMP reduced to 1.3 during test SC2 - 400%, as damage of the piers limited the floor accelerations to values smaller than the ones reached in the previous run.

The roof acceleration followed a different trend: prior to gable cracking the ratio was nearly constant around 2.3, while after gable cracking (test SC2 - 100%) it dropped below 2.0. The roof AMP raised back to 2.5 in test SC2 - 300%, but major gable damage and connection failures prevented further increase of the roof acceleration: during run SC2 - 400% the roof AMP reduced to 2.2. A similar but less pronounced behaviour was recorded at midspan of the floor diaphragm, due to interaction with the out-of-plane response of the gables through the Y1 anchors (Fig. 5c). In this case, the initial value of about 1.5 dropped below 1.3 starting from test SC2 - 100%, and exceeded 1.3 only during run SC2- 300%.

The first-mode period progressively elongated from 0.1 s to 0.22 s (Fig. 18). The first significant change was observed at the end of test SC2 - 100%, when the period increased to 0.13 s. No damage was reported following this run, which was associated with the DL1 threshold; however, minor cracks might have formed and completely closed back at the end of the motion. Further significant elongation was noticed after test run SC2 - 300%, when the period reached 0.18 s at DL3 limit state. The building in its final damage conditions was characterized by a fundamental period of 0.22 s.

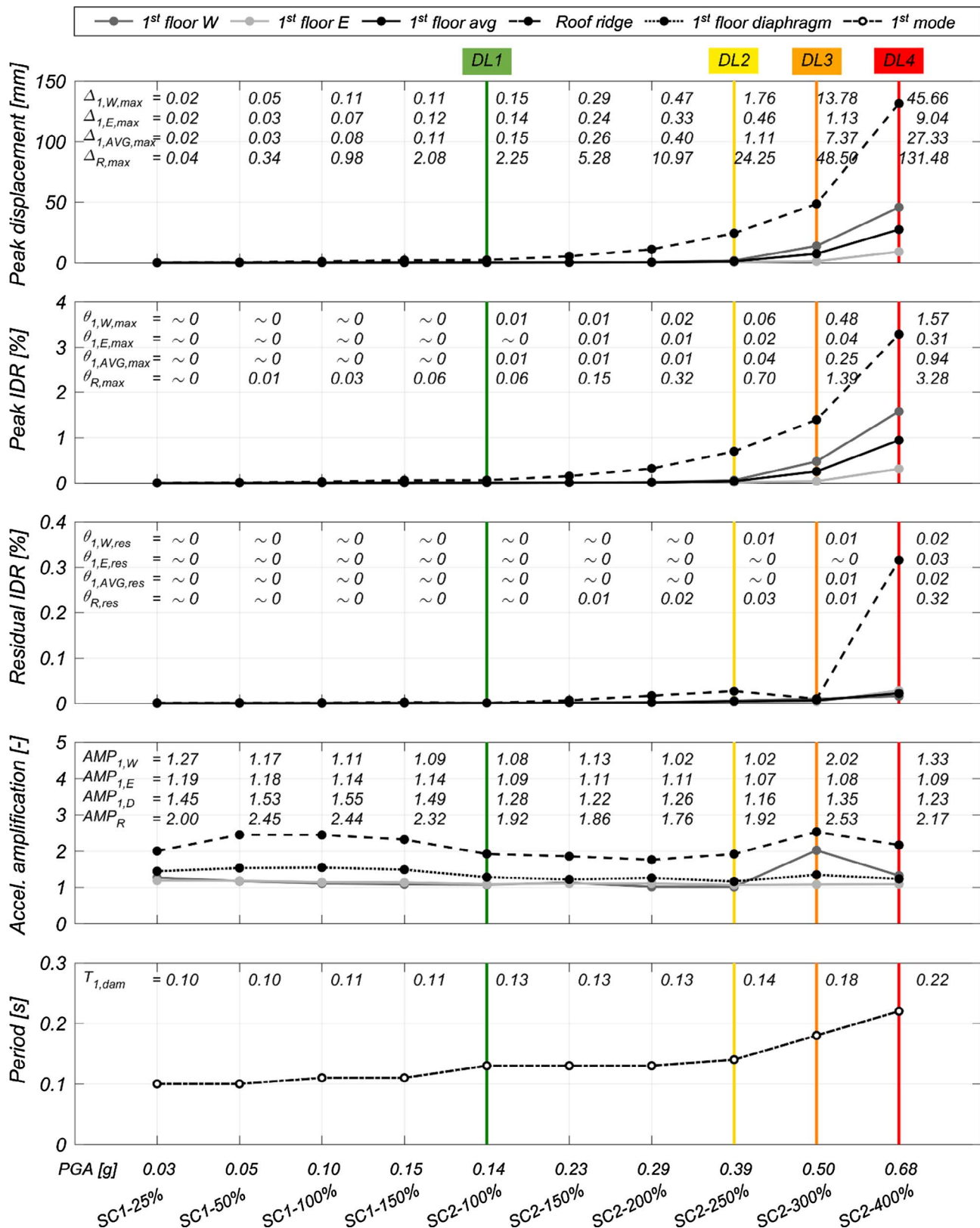


Fig. 18. Summary of the performance of the building specimen.

7. Conclusions

A full-scale unidirectional shake-table test was conducted on a prototype clay-brick URM detached house with flexible diaphragm and high gables, featuring typical details of the Dutch pre-1940s

construction practice. The specimen was subjected to incremental dynamic excitations, with input motions representative of induced seismicity scenarios for the Groningen region in the Netherlands, characterized by smooth response spectra and short significant duration. The ground motions were progressively scaled in amplitude to achieve

the desired demand intensities. The building suffered only minor damage up to the input motion with PGA of 0.23 g and reached its near-collapse state at a PGA of 0.68 g. The test results showed significant out-of-plane damage in zones of high acceleration response, such as the roof gables. Damage due to wall in-plane response was instead mostly associated with rocking of slender piers.

The test outcomes confirm that gable walls are the most vulnerable components of this type of structures: at the end of the test sequence, the most severe damage was related to their out-of-plane overturning and the significant residual dislocation of the North half-hipped gable. The gables-roof assembly exhibited very flexible response, experienced displacement demands significantly larger than at lower levels, and reached near-collapse conditions. Good interlocking between intersecting walls resulted in the participation of longitudinal wall portions to the gable mechanisms. Weak connections between roof purlins and gables and between wall plates and longitudinal walls did not cause significant differential displacements at these interfaces; however, damage occurred at the nailed connection between roof trusses and wall plates.

Despite having the same overall pier cross-sectional areas, the two longitudinal façades exhibited different vulnerabilities. Significant damage occurred in the West wall, where slender piers developed flexural-rocking in-plane mechanisms and a lintel nearly collapsed during the final stages of the test. The East side exhibited a stiffer and stronger response than the West one, because it incorporated a longer squat pier. Because of differential longitudinal displacements between the East and West walls the floor diaphragm underwent in-plane shear deformations, but its high flexibility prevented overall torsional response of the building.

Increasing progressively the seismic demand imposed on the prototype ensured the completion of a large number of dynamic tests before reaching near-collapse conditions. These tests provide a unique data set that captures at full scale the in-plane and out-of-plane behaviour of URM walls, and the influence of flexible diaphragms on the global dynamic response of entire buildings. Interpretation of the experimental results will constitute the basis for the development of analytical and numerical models, to estimate the dynamic response and the parameters for the performance-based seismic assessment of URM buildings. All recorded data (acceleration and displacement time histories) and the videos of the tests can be requested online on www.eucentre.it/nam-project.

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Appendix A. Supplementary material

Supplementary data associated with this article can be found, in the online version, at <http://dx.doi.org/10.1016/j.engstruct.2018.02.016>.

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