COLLAPSE SHAKE-TABLE TEST ON A URM-TIMBER ROOF SUBSTRUCTURE

António A. CORREIA¹, Umberto TOMASSETTI², Alfredo CAMPOS COSTA³, Andrea PENNA⁴, Guido MAGENES⁵, Francesco GRAZIOTTI⁶

ABSTRACT

Typical low-rise masonry construction in regions such as Europe, Australia and New Zealand consists of cavity or solid URM walls covered with various timber roof configurations generally supported by masonry gables. The flexibility of these light timber roof systems has been reported to have significant effect on the seismic performance of the structure as a whole. Post-earthquake observations and experimental outcomes highlighted the large vulnerability of the URM gable walls to the development of overturning mechanisms, both due to the out-of-plane excitations and the in-plane timber diaphragm deformability. This paper presents a full-scale collapse shake-table test on a Dutch roof substructure composed by wooden planks supported by timber beams lying on masonry gable walls. After attaining the collapse of the gable walls, the timber roof diaphragm was subjected to a supplementary quasi-static cyclic pushover test for a complete characterization of the system response. The test is a part of a wider research project aimed at assessing the vulnerability of URM buildings in Groningen, a region of the Netherlands not naturally prone to seismic events, but which has recently been exposed to induced seismicity. The presented results include the damage evolution, the collapse mechanism and the hysteretic response of the specimen.

Keywords: Full-scale shake-table test; Unreinforced masonry; Timber; Roof; Collapse

1. INTRODUCTION

Unreinforced masonry (URM) buildings represent about 90% of the building stock in the Groningen region of the northern Netherlands (Crowley and Pinho 2017). Although they were not specifically designed for seismic actions, they have been exposed to low-intensity shakings due to seismic events induced by gas extraction and consequent reservoir depletion in recent years (Bourne et al. 2015). For that reason, an experimental campaign was launched in 2015 for characterizing the performance of structural components, assemblies, and systems with the aim of improving the 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 (Tondelli et al. 2015, Zapico et al. 2018) and laboratory tests, such as: (i) characterization tests on bricks, mortar and small masonry assemblies; (ii) in-plane cyclic shear-compression tests (Graziotti et al. 2016a) and dynamic out-of-plane tests on full-scale masonry piers (Graziotti et al. 2016b); and (iii) full-scale unidirectional and bidirectional shake-table tests on different URM building typologies (Graziotti et al. 2017, Tomassetti et al. 2017, Graziotti et al. 2018, Kallioras et al. 2018).

Terraced houses represent more than 50% of the URM building stock of the region. Most terraced houses...
are built with cavity walls, consisting of two leaves of bricks, possibly with insulating material in between. The inner leaf has load-bearing function and is usually made of calcium silicate (CS) bricks, while the outer leaf is often a clay-brick veneer with only aesthetic and weather-protection function. The two leaves are usually connected by regularly distributed steel 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, testing an end unit of an entire terraced house is representative of its seismic response. A uniaxial shake table test was thus carried out in the EUCENTRE laboratory on a full-scale two-story building, with timber roof and RC slabs, representing an end-unit of a typical terraced house (Graziotti et al. 2017). A second shake table test was performed at the LNEC laboratory on a full-scale sub-volume of the EUCENTRE specimen, corresponding to its second floor and roof. This test was biaxial, including a seismic excitation in the vertical direction besides the horizontal one (Tomassetti et al. 2017). Nevertheless, the interest in characterizing the response of the gable walls and roof system led to the decision to build a third specimen related to a typical terraced house, focusing only on its roof substructure, which is the subject of this paper.

1.1 Description of the roof substructure

The 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. 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 two gable walls in the transverse façades supported the roof beams of the timber roof, which were mechanically connected to the CS walls. 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 1 presents the plan view at the base of the specimen and its North-East and South-West views, while Figure 2 shows the elevation views of the specimen. The blue dots indicate the locations of the steel ties connecting the two leaves.

![Figure 1. Plan view at the base of the specimen and its North-East and South-West views.](image1)

![Figure 2. Elevation views of the specimen (dimensions in cm).](image2)

The pitched timber roof, with 42° of inclination, is a simple structure consisting of one ridge beam, two
timber plates on the sides of the RC slab and two girders per side between the ridge beam and the timber plates, at approximately 1.2 m of distance. The timber plates, belonging to the roof but positioned at the slab level, are attached to the longitudinal sides of the slab by means of 100 cm-spaced 10 mm-diameter threaded bars cast into the RC slab. Tongue and groove planks, with a width equal to 182 mm and a thickness of 18 mm, were nailed on top by means of two 60x2 mm nails at each intersection, as foreseen in Figure 3. The in-plane stiffness of the timber roof diaphragms is essentially provided by the nailed connections between beams and planks, as well as by the effectiveness of the tongue and groove joints.

Figure 3. Geometry and details of the timber roof diaphragms (dimensions in cm).

1.2 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.

Table 1. Material properties.

<table>
<thead>
<tr>
<th>Material property [units]</th>
<th>Brick type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CS</td>
</tr>
<tr>
<td>Density of masonry [kg/m³]</td>
<td>1796</td>
</tr>
<tr>
<td>Brick compressive strength [MPa]</td>
<td>18.72</td>
</tr>
<tr>
<td>Mortar compressive strength [MPa]</td>
<td>3.70</td>
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<tr>
<td>Mortar flexural strength [MPa]</td>
<td>2.50</td>
</tr>
<tr>
<td>Masonry compressive strength [MPa]</td>
<td>7.03</td>
</tr>
<tr>
<td>Masonry Young’s modulus [MPa]</td>
<td>6090</td>
</tr>
<tr>
<td>Masonry flex. bond strength [MPa]</td>
<td>0.33</td>
</tr>
</tbody>
</table>

2. INSTRUMENTATION AND TESTING PROTOCOL

Several sensors were installed on the specimen to monitor its response: 28 accelerometers, 9 wire potentiometers, and 12 LVDTs. A rigid steel-frame was installed in the interior of the specimen as a reference system for direct measurements of the displacements. The specimen was subjected to incremental unidirectional dynamic tests, applying a series of shake-table motions of increasing intensity. The acceleration histories replicated the second floor motion of the full-scale two-story building shake table test carried out in the EUCENTRE laboratory (Graziotti et al. 2017). The original ground accelerograms were characterized by smooth response spectra and they were proposed by Bommer et al. 2015. The target spectra, shown in Figure 4, reflect the frequency content evolution with damage of the original model, enlarging the quasi-constant acceleration plateau with respect to the associated ground motion. Table 2 presents the testing sequence on the roof substructure and both the target and recorded table motion peak values. The tests were conducted up to collapse of the gable walls.
Figure 4. Theoretical horizontal 5% damped acc. response spectra of the experimental inputs.

Table 2. Summary of the testing sequence.

<table>
<thead>
<tr>
<th>Test Input</th>
<th>Scale factor [%]</th>
<th>Associated PGA [g]</th>
<th>Nominal PTA [g]</th>
<th>Nominal $S_a(T_1)$ [g]</th>
<th>Recorded PTA [g]</th>
<th>Calculated $S_a(T_1)$ [g]</th>
<th>Calculated PTV [mm/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEQ1</td>
<td>50</td>
<td>0.050</td>
<td>0.066</td>
<td>0.102</td>
<td>0.074</td>
<td>0.105</td>
<td>34.9</td>
</tr>
<tr>
<td>FEQ1</td>
<td>100</td>
<td>0.099</td>
<td>0.132</td>
<td>0.203</td>
<td>0.143</td>
<td>0.205</td>
<td>64.5</td>
</tr>
<tr>
<td>FEQ1</td>
<td>150</td>
<td>0.137</td>
<td>0.168</td>
<td>0.296</td>
<td>0.170</td>
<td>0.286</td>
<td>95.0</td>
</tr>
<tr>
<td>FEQ2</td>
<td>50</td>
<td>0.087</td>
<td>0.102</td>
<td>0.154</td>
<td>0.106</td>
<td>0.182</td>
<td>61.9</td>
</tr>
<tr>
<td>FEQ2</td>
<td>100</td>
<td>0.170</td>
<td>0.204</td>
<td>0.309</td>
<td>0.207</td>
<td>0.312</td>
<td>136.0</td>
</tr>
<tr>
<td>FEQ2</td>
<td>150</td>
<td>0.243</td>
<td>0.284</td>
<td>0.341</td>
<td>0.245</td>
<td>0.380</td>
<td>160.2</td>
</tr>
<tr>
<td>FEQ2</td>
<td>200</td>
<td>0.307</td>
<td>0.316</td>
<td>0.483</td>
<td>0.487</td>
<td>0.640</td>
<td>274.1</td>
</tr>
<tr>
<td>FEQ2</td>
<td>300</td>
<td>-</td>
<td>0.472</td>
<td>0.725</td>
<td>0.668</td>
<td>0.774</td>
<td>415.3</td>
</tr>
<tr>
<td>FEQ2</td>
<td>400</td>
<td>-</td>
<td>0.633</td>
<td>0.966</td>
<td>0.935</td>
<td>1.138</td>
<td>538.3</td>
</tr>
<tr>
<td>FEQ2</td>
<td>500</td>
<td>-</td>
<td>0.791</td>
<td>1.208</td>
<td>0.955</td>
<td>1.091</td>
<td>614.8</td>
</tr>
<tr>
<td>FEQ2</td>
<td>600</td>
<td>0.170</td>
<td>0.204</td>
<td>0.309</td>
<td>0.201</td>
<td>0.339</td>
<td>128.3</td>
</tr>
</tbody>
</table>

PGA = peak ground acceleration; PTA = peak table acceleration; PTV = peak table velocity; $S_a(T_1)$ = spectral acceleration (5% damping) at fundamental period.

3. SHAKE TABLE TEST RESULTS

The following sections illustrate the performance of the specimen, reporting qualitative damage observations, collapse mechanism description, evolution of dynamic properties, and response plots. At the end of every shaking test stage the structural damage was surveyed in detail, while videos of the testing sequences are available on the youtube.com EUCLINÉS’ channel.

3.1 Damage evolution

The outside of the East CS wall was covered with a white plaster layer, making the detection of new cracks easier. Figure 5 shows the evolution of the damage surveyed on both gable walls throughout the entire testing sequence.

The first visible damage associated with a shake table motion was detected during test FEQ1-100% ($S_a(T_1) = 0.205$ g). 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 5. 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 – see section 3.3.

There was a crack opening at the base the CS East wall during test FEQ2-100% ($S_a(T_1) = 0.312$ g), 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 leafs and a crack was observed at the outer clay wall.

Test FEQ2-150% caused no new damage on the structure, while FEQ2-200% (Sa(T1) = 0.640 g) 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% (Sa(T1) = 0.774 g), 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% (Sa(T1) = 1.138 g). On the other hand, the latter test induced an enlargement of the cracks on the East wall, interconnecting several of the pre-existing ones. At this point, several instruments were removed.

Afterwards, test stage FEQ2-500% (Sa(T1) = 1.091 g) 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% (Sa(T1) = 1.410 g). 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%, as described in the following section.

3.2 Description of the collapse mechanism

Figure 6 depicts a sequence of frames of the video when the building prototype exhibited a local collapse of the East wall. Figure 6 (a) to (d) represent a first impulse, with the gable wall bending to the interior of the model. The crack at the base of the wall opened significantly due to flexure/rocking, followed by the opening of the horizontal crack between the 2/3 height roof beams. In Figure 6 (d) the motion was opposite, with the closure of those cracks and the opening of the vertical crack at the upper third of the gable which moved outward of the model.

Figure 6 (e) to (g) represent a second impulse of the gable towards the interior of the model, with the two horizontal cracks opening very significantly and leading to the formation of the vertical crack at half span and in the lower 2/3 of the wall, as visible in Figure 6 (f). The collapse mechanism was then completely formed and that portion of the East gable wall had a full collapse towards the interior of the model, as illustrated in Figure 6 (g).

![Diagram of crack pattern](Image)

Figure 5. Evolution of the crack pattern in the gable walls along the test stages.

Afterwards, the upper third of the gable wall, fully cracked and with complete loss of connection to the
roof beams, also collapsed. The timber roof had a flexural response with a deflection towards the interior of the model, since it had no longer any support on the East side of the model, as shown in Figure 6 (h) to (j). Figure 7 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.

Figure 6. Snapshots of the FEQ2-600% test.
3.3 Evolution of dynamic properties

The evolution of the fundamental frequency of vibration of the specimen and of the corresponding modal damping is summarized in Table 3. These results show that the dynamic characteristics of the specimen, in terms of its first mode of vibration, had a consistent evolution with the increasing intensity of the test stages. It should be noted that the estimation of modal damping values is significantly more uncertain than the one of modal frequencies, but there is a clear trend for an increase in the first mode damping values.

Table 3. Evolution of the fundamental mode of vibration of the model during the shake table test.

<table>
<thead>
<tr>
<th>Dynamic identification test</th>
<th>Frequency [Hz]</th>
<th>Damping [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (initial state)</td>
<td>12.08</td>
<td>2.07</td>
</tr>
<tr>
<td>2 (after FEQ1-100%)</td>
<td>12.10</td>
<td>2.02</td>
</tr>
<tr>
<td>3 (a. FEQ1-150%)</td>
<td>11.74</td>
<td>2.07</td>
</tr>
<tr>
<td>4 (a. FEQ2-100%)</td>
<td>11.22</td>
<td>2.17</td>
</tr>
<tr>
<td>5 (a. FEQ2-150%)</td>
<td>11.14</td>
<td>1.81</td>
</tr>
<tr>
<td>6 (a. FEQ2-200%)</td>
<td>10.56</td>
<td>2.30</td>
</tr>
<tr>
<td>7 (a. FEQ2-300%)</td>
<td>10.46</td>
<td>1.83</td>
</tr>
<tr>
<td>8 (a. FEQ2-400%)</td>
<td>10.23</td>
<td>2.41</td>
</tr>
<tr>
<td>9 (a. FEQ2-500%)</td>
<td>10.23</td>
<td>2.35</td>
</tr>
</tbody>
</table>

3.4 Hysteretic response

The evolution of the specimen’s hysteretic response is shown in Figure 8, in terms of base shear, $V$, versus roof diaphragm drift, $\gamma_R$, for each test stage. The histories of the base shear have been computed as the sum of the products of acceleration recordings times the tributary mass of the corresponding accelerometer. Masses are assumed to be lumped at the accelerometer locations. The dynamic force-displacement backbone curve, also represented in Figure 8, is defined as the plot of the maximum base shear, $V_{\text{max}}$, and the corresponding relative drift for each stage of testing. The last point of both the positive and negative branch was obtained as the pair of the maximum drift attained and the corresponding base shear. The attainment of the higher base shear occurred for sway towards the negative direction (towards the single-leaf side, East), while the higher drift was in the opposite direction (towards the double-leaf side, West).

3.5 Damage Limit States

In this section, the identification of global quantitative thresholds that adequately describe the overall structural damage state of the building is attempted. Six damage states (DSs) were considered: DS$_0$, completely undamaged; DS$_1$, no structural damage; DS$_2$, minor structural damage; DS$_3$, moderate structural damage; DS$_4$, extensive structural damage; and DS$_5$, very heavy structural damage, total or local collapse.
The damage limits (DLs), defining quantitative boundaries in terms of inter-story drift between the aforementioned damage states, were defined as follows:

- **DL₀** corresponds to the maximum achieved level of displacement with no visible damage (structural or non-structural). It was identified with a diaphragm drift equal to 0.03%, achieved during FEQ1-150% \((S_{u(T1)}=0.286 \text{ g}; \text{PTA}=0.170 \text{ g}; \text{PTV}=95.0 \text{ mm/s})\) which did not cause any further damage;

- **DL₁** corresponds to the maximum achieved level of displacement with no visible structural damage. After test FEQ2-100% \((S_{u(T1)}=0.312 \text{ g}; \text{PTA}=0.207 \text{ g}; \text{PTV}=136.0 \text{ mm/s})\), a minor crack was detected on the plaster layer at the base of the East gable;

- **DL₂** is defined as the maximum achieved level of displacement with minor/slight structural damage. It was identified at the end of test FEQ2-200% \((S_{u(T1)}=0.640 \text{ g}; \text{PTA}=0.487 \text{ g}; \text{PTV}=274.1 \text{ mm/s})\), when the complete cracking of the bottom layer of the East gable occurred. The recorded peak diaphragm drift was equal to 0.26%;

- **DL₃** is defined as the maximum achieved level of displacement with moderate structural damage (but still repairable). This state was associated with FEQ2-400% \((S_{u(T1)}=1.138 \text{ g}; \text{PTA}=0.935 \text{ g}; \text{PTV}=538.3 \text{ mm/s})\), upon the development of several cracks on the East gable due to a peak diaphragm drift equal to 1.92%;

- **DL₄** corresponds to the maximum achieved level of displacement with extensive structural damage (i.e., not repairable). The limit can be considered as a collapse-prevention threshold. DL₄ was attained after a peak diaphragm drift of 3.55%, recorded during test FEQ2-500% \((S_{u(T1)}=1.091 \text{ g}; \text{PTA}=0.955 \text{ g}; \text{PTV}=614.8 \text{ mm/s})\), and was associated with heavy damage on the East gable and cracking on the West gable wall;

- **DL₅** equals the displacement associated with the failure of the East gable. It was identified with a peak diaphragm drift of 5.65% during FEQ2-600% \((S_{u(T1)}=1.410 \text{ g}; \text{PTA}=1.138 \text{ g}; \text{PTV}=677.9 \text{ mm/s})\).

Figure 9 identifies the DLs on the experimental backbone curve defined in terms of the base shear coefficient and the diaphragm drift. The base shear coefficient, \(BSC\), is defined as the ratio of the base shear and the total weight of the specimen (excluding the reinforced concrete base).

### 4. PUSHOVER TEST RESULTS

Following the shake table test on the complete roof substructure, the remaining URM cavity wall was carefully removed and a support timber structure was put in place to allow for additional testing on the timber roof system. This support structure was assembled in order to restore the timber roof system's
geometry and to carry its weight, while also serving as a guidance system for the timber roof, as depicted in Figure 10, with the use of PTFE plates between the timber surfaces. A cyclic pushover test could thus be performed using the shake table as actuator system and taking advantage of the particular characteristics of LNEC’s shake table platform, which is surrounded by three reaction walls. The East and West extremities of the ridge beam were fixed to the reaction walls through steel ties, as shown in Figure 10.

Both steel ties were instrumented with load cells, while each of the five timber roof beams was instrumented with a wire potentiometer to measure its horizontal displacement with respect to the reference steel frame.

The control system of the shake table platform was prepared for using the relative displacement between the reference frame and the ridge beam as control variable, thus ensuring that the desired drift on the specimen was applied at each cycle. The roof structure was then subjected to two full cycles at ±10 mm, ±50 mm, ±100 mm and ±150 mm. The force-diaphragm drift curve obtained is represented in Figure 11.

It can be observed that the response was composed of three different stages: (i) an initial frictional response with a limit of about ±3 kN; (ii) followed by a low-stiffness response corresponding to a sliding associated to the limits of the nails' free-deformation in their holes and to a relative distortion between the different timber planks; (iii) and a stiffening response associated to the dowel behavior of the nails.

![Figure 9. Identification of the DLs on the roof backbone curve.](image9)

![Figure 10. Support and guidance system and steel ties connecting the ridge beam to the reaction walls.](image10)
No particular damage was observed during the cyclic loading up to 150 mm, with the observed response being rather stable. Some cyclic strength degradation occurred in the second cycles, attributed to the ovalization of the nailed connections and to partial pull-out of the nails.

It was also found that the evolution of displacements between the different roof beams remained linearly proportional to their elevation, despite the load being applied at the ridge beam only. This indicates that each roof plan is deforming as a shear column and not with in-plan flexure.

After the cyclic loading, the roof was pushed up to 250 mm and unloaded. A permanent drift was visible, amounting to around 235 mm. The timber planks distorted, ones with respect to the others, and some nails at the base beams started to pull-out, opening gaps in their connections. A significant increase of broken tiles occurred on the South side. No broken tiles appeared on the North side, since the mirrored configuration of the tiles was particularly fragile on the South side in the direction of pushing – the damage to the tiles seems to have no relation with the existence or not of a window opening.

Finally, the near collapse limit state was reached for a pushover above 350 mm, see Figure 12 (a,b). The nails connecting the timber planks and the base beams were completely pulled out, as depicted in Figure 12 (c-f). The roof structure had a significant permanent drift of over 400 mm, as represented in the load-diaphragm drift plot of Figure 11 and shown in Figure 12 (g). However, the readings of the horizontal displacement above 370 mm were assumed to be non-realistic since the wire of the potentiometer was inclined due to the uplift of the roof structure.

The damage to the clay tiles on the South side is documented in Figure 12 (h), with broken and uplifted tiles visible throughout the roof plan but particularly at midspan. Figure 12 (i) allows visualizing the shear deformation of the roof plan due to distortion of the timber planks through sliding at their tongue and groove joints.
Figure 12. Views of the damaged specimen at the end of the pushover phase: (a,b) images of the damage during pushover; nails pull-out after the pushover on the (c,d) West side and (e,f) East side; (g) permanent drift; (h) cracked roof tiles; (i) distortion of timber planks.

5. CONCLUSIONS

This paper presented the experimental tests carried out to characterize the seismic response of a URM-timber roof substructure, in order to complement previous shake table tests on full-scale models representative of typical terraced houses of the Groningen region. 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. It denoted that the specimen was not significantly affected in its strength capacity and in its stiffness up to relatively high level of seismic excitation (PGA ≈ 0.3g), although the gradual formation of a crack pattern leading to a failure mechanism in the gables was observed when the specimen was subjected to a motion larger than 1g.
The roof system was tested on the shake table up to the out-of-plane collapse of its URM gable walls, after which the remaining timber roof structure was tested quasi-statically for a horizontal load applied at its ridge beam. In fact, once the hysteretic response of the complete specimen was known, it was deemed interesting to assess the load-displacement behavior of the timber roof system alone, in order to better characterize the influence of the URM gable walls on the specimen’s seismic response. From the cyclic pushover tests, it was found that the timber roof structure presented a stable response up to drifts that double the ones attained in the shake table test and causing the failure of the gable walls. Moreover, the maximum load attained reached about 40 kN, which corresponds to about 90% of the overall load-carrying capacity of the roof substructure.

6. ACKNOWLEDGMENTS

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7. REFERENCES


