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MONOTONIC, CYCLIC AND DYNAMIC BEHAVIOUR OF TIMBER-MASONRY CONNECTIONS

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ABSTRACT: Historical or existing buildings are often composed of brick or stone masonry walls, and timber floors and roofs. When these buildings are subjected to earthquakes, the interaction among such structural components is essential to avoid collapse or excessive damage to the constructions. In this framework, a crucial role is played by the connections between horizontal and vertical structural elements: this is especially true when considering existing buildings not designed or realized taking into account seismic actions, because no measures against these events are present. An example of such a situation is noticeable in the province of Groningen, in the northern part of the Netherlands, where human-induced earthquakes take place due to gas extraction. Given the absence of seismic events until recently, a characterization of the existing buildings is necessary, and strengthening measures have to be analysed, in order to make them earthquake-safe. This work describes the experimental campaign conducted at Delft University of Technology to assess the seismic behaviour of as-built and strengthened timber-masonry connections. Results show that existing joints are not suitable to withstand earthquakes, while the proposed strengthened configurations can increase not only strength and stiffness, but also energy dissipation of the connections.

KEYWORDS: Timber-masonry connections, Seismic response, Existing buildings, Retrofitting

1 INTRODUCTION

Due to gas extraction, induced earthquakes have recently started to take place in the region of Groningen, in the northern part of the Netherlands. These events were unknown until a few years ago, and therefore the current building stock is not suitably designed to withstand seismic actions. These buildings are mainly composed of unreinforced single-leaf or double-wythe brick masonry walls, and timber floors and roofs. The slenderness of the walls and the high in-plane flexibility of the diaphragms, due to small-size timber structural elements, make existing buildings very vulnerable against earthquakes, although the magnitude of these events has not been very high (M_L = 3.6) until now. However, the absence of relevant past earthquakes also constitutes a significant source of uncertainty: the maximum future seismic actions on structures are unknown, and could be much more intense than those already occurred.
Therefore, a characterization of as-built structural components took place, and retrofitting measures were defined on this basis: within this seismic assessment, a full-scale experimental campaign was arranged and conducted at Delft University of Technology, focusing on the seismic response of as-built and strengthened timber-masonry connections. Such a focus was necessary because, although the interaction of timber floors and masonry walls through the joints among them plays a crucial role in the seismic response of existing buildings, limited data are available.
For the US context, in [1] the results of an experimental campaign on as-built timber-masonry connections were reported. The samples represented either a joist embedded in a masonry pocket with mortar, or the same configuration with the additional presence of a nailed anchor. Three test types were performed: monotonic, quasi-static cyclic, and dynamic cyclic. The latter loading case consisted of a high-frequency repetition of ten cycles at the same amplitude. This signal was then scaled to describe the full response of the connection. The samples were very compact, and it was therefore not possible to assess the response of the whole wall portion around the joint, while a more detailed characterization of the frictional response and the failure modes of nails and anchors was conducted. Due to the specific configuration of the connection, the detected response was asymmetric: when pulling the joist, only the nails and the anchor could resist the load, while in the opposite direction also the masonry bricks played a role, as reported in [1].
With regard to the European context, as-built and retrofitted wall-to-floor connections with typical Portuguese features were studied in [2,3] by performing monotonic and cyclic pull-out tests on full-scale samples. The strengthening solution consisted of a tie rod anchoring the wall to the joist through a steel angle. Since several components were present, their various failure modes were identified and discussed. In this case, due to the large dimensions of the tested sample, the

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effect of the masonry portion around the floor joist could be addressed, as well as the influence of the wall thickness.

Finally, in-situ tests on two unreinforced masonry buildings in New Zealand were performed and their outcomes discussed in [4]. Monotonic loading was applied to plate anchor and timber blocking connection types, and a thorough discussion of the failure modes was carried out. Timber joist splitting was the most commonly observed failure mode for the anchor plate connection, showing that its capacity was mainly governed by the condition of the timber joist or the characteristics of the wall. Timber blocking connection showed great ductility, but in general less capacity compared to plate anchor one.

Starting from the knowledge of previous research studies, for the campaign described in this work it was chosen to broaden the information that could be retrieved from the tests. Therefore, the size of the samples was defined in such a way that it was possible to study the behaviour of the portion of wall around the timber joist; additionally, dynamic tests with a signal of an induced Groningen earthquake were performed, besides the usual monotonic and quasi-static cyclic ones. This led to a thorough characterization of the seismic behaviour of the analysed timber-masonry connections.

2 MATERIALS AND METHODS

2.1 MATERIALS

Two as-built and five strengthened connection types, consisting of a 980×600×100 mm single-leaf clay brick masonry wall and a 65×170 mm timber joist placed orthogonally to it, were tested under monotonic, quasi-static cyclic and dynamic loading. Joists and, where present, planks were made of C24 [5] spruce (*Picea Abies*) timber, while the main material properties of clay brick masonry are reported in Table 1.

Figure 1 shows the seven tested configurations. The two as-built ones represented a simple masonry pocket (A), and a joint realized with a 240×240×14 mm hook anchor (B), fastened to the joist with three 4×55 mm nails.

The first proposed refurbishment (C) consisted of a steel angle fastened to the joist with four 5×60 mm screws and to the masonry with two 10×95 mm mechanical anchors. Option D consisted of a further 80×80 mm joist fastened below the existing one by means of two 90×90×3 mm steel brackets and four 5×60 mm screws for each of them, and attached with two 10×165 mm mechanical anchors to the masonry.

Table 1: *Main material properties of clay brick masonry; in parentheses the coefficient of variation is shown*

Parameter	Average value
Masonry compressive strength f'_m (MPa)	11.87 (0.09)
Masonry elastic modulus E_m (MPa)	3278 (0.17)
Flexural bond strength f_w (MPa)	0.11 (0.51)
Initial shear strength f_{v0} (MPa)	0.15 (0.10)
Shear friction coefficient μ	0.78 (0.10)

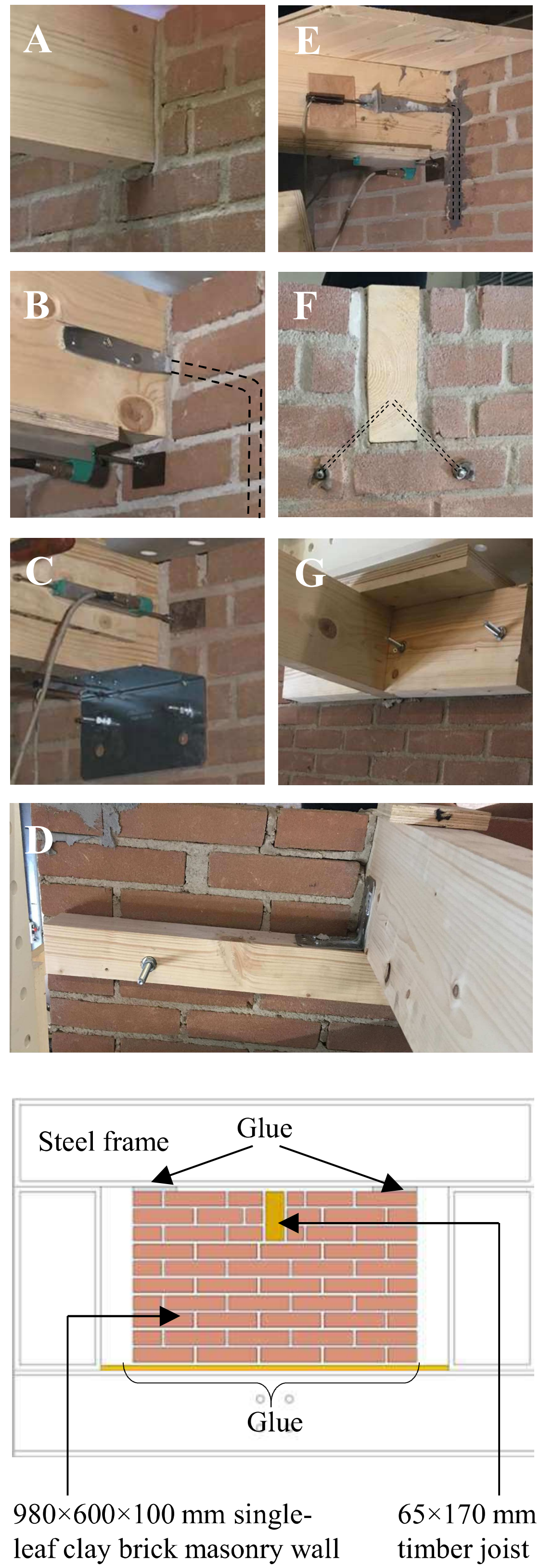


Figure 1: *Tested timber-masonry connections (A-G) and typical specimen's configuration, including boundary conditions*

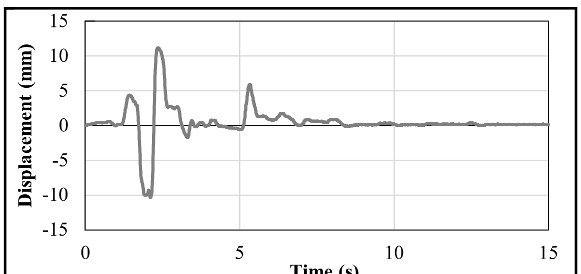
Configuration E consisted of a 240×240×14 mm hook anchor fastened to the joist with three 4×55 mm nails, and then glued with epoxy to a 25×40×240 mm incision realized on the wall.
In option F the joist was connected to the wall with two inclined 7×180 mm screws. These were inserted into the joist after drilling holes having a 10 mm diameter in the masonry: these holes were then filled with injected epoxy; the screws were therefore partly embedded in the glue and partly inserted in the joist.
Retrofitting method G was part of a floor strengthening presented in [6], and was realized with 65×170 mm timber blocks placed on both sides of the joist (in practice they would be placed between each couple of joists). The blocks were firstly fixed to the existing joist by means of two 5×70 mm screws drilled at an angle of 45 degrees, and then fastened to the masonry with two 10×165 mm mechanical anchors each. Because this intervention would in practice involve also the timber diaphragm, it was important to recreate the same conditions: hence, besides the presence of 18×165 mm planks, fixed to the joist with two 3×65 mm nails, also an additional 18-mm-thick plywood panel was fastened to them and inside the blocks with five 5×70 mm screws.
The boundary conditions of the samples are shown in Figure 1 as well: they were surrounded by a steel frame, and glued on the whole bottom side and at the two top corners, in order to both study the behaviour of the connection as such, and to include the influence of the masonry around it. The tests aimed to simulate connections at the roof level, identified as one of the most vulnerable parts for Groningen buildings; the configurations to be tested were defined in agreement with consultants. Further details can be found in [7].

2.2 METHODS

For each configuration described in section 2.1, seven tests were performed: a single monotonic test (M) to determine the ultimate displacement, three quasi-static reversed-cyclic tests (QS), and three high-frequency dynamic tests (HFD), as reported in Table 2. Thus, in total 49 samples were tested.
Monotonic and quasi-static reversed-cyclic tests were performed according to ISO 16670 [8]. Therefore, after the determination of the ultimate displacement of each configuration through monotonic tests, the amplitudes of the steps were defined accordingly. Furthermore, each step consisted of three cycles at 0.3 mm/s rate, in agreement with the standard.
Dynamic tests were performed by applying to the specimens a specific high-frequency signal generated by the hydraulic actuator. In this way, a comparison to quasi-static test results was possible, and the effect on the connection of a short, sudden signal, such as a typical Groningen induced earthquake, could be analysed. The input dynamic signal was chosen starting from performed shaking table tests on full-scale typical Dutch buildings: it consisted of a recorded displacement history of a timber-masonry connection during a shaking table test performed at Eucentre [9], and it is shown in Figure 2 together with the performed runs. This reference signal was induced with an input accelerogram corresponding to 133% of the estimated reference response spectrum of Groningen region, for a return period of 2475 years [9].
For these tests on connections, a procedure similar to nonlinear incremental dynamic analyses was followed, starting from a very small signal's peak amplitude (0.25 mm), and then repeating it until collapse or maximum applied displacement (25 mm) was reached.

Table 2: *List of performed tests and adopted samples' names*

Joint	Test types	Sample name(s)
	1 monotonic	A-M-1
A	3 cyclic	A-QS-1, A-QS-2, A-QS-3
	3 dynamic	A-HFD-1, A-HFD-2, A-HFD-3
	1 monotonic	B-M-1
B	3 cyclic	B-QS-1, B-QS-2, B-QS-3
	3 dynamic	B-HFD-1, B-HFD-2, B-HFD-3
	1 monotonic	C-M-1
C	3 cyclic	C-QS-1, C-QS-2, C-QS-3
	3 dynamic	C-HFD-1, C-HFD-2, C-HFD-3
	1 monotonic	D-M-1
D	3 cyclic	D-QS-1, D-QS-2, D-QS-3
	3 dynamic	D-HFD-1, D-HFD-2, D-HFD-3
	1 monotonic	E-M-1
E	3 cyclic	E-QS-1, E-QS-2, E-QS-3
	3 dynamic	E-HFD-1, E-HFD-2, E-HFD-3
	1 monotonic	F-M-1
F	3 cyclic	F-QS-1, F-QS-2, F-QS-3
	3 dynamic	F-HFD-1, F-HFD-2, F-HFD-3
	1 monotonic	G-M-1
G	3 cyclic	G-QS-1, G-QS-2, G-QS-3
	3 dynamic	G-HFD-1, G-HFD-2, G-HFD-3



Run #	% scaling of reference signal	Run #	% scaling of reference signal
1	2.5	9	60
2	5	10	70
3	7.5	11	80
4	10	12	90
5	20	13	100
6	30	14	150
7	40	15	200
8	50	16	240

Figure 2: *Reference signal adopted for high-frequency dynamic tests; the various runs and their amplitudes are also shown*

In order to perform all presented tests, a versatile test setup had to be designed; this is shown in Figure 3, together with the adopted measurement plan. The setup was composed of a fixed part, on which a rolling structure was placed, enabling the movement of the sample provided by a hydraulic actuator.

Rotations of the moving part were prevented by proper plates and wheels close to the rollers, and allowing only the axial horizontal displacements transmitted by the actuator. Every sample was surrounded by a steel frame, to guarantee its stability; this frame was also furtherly connected with bracings to the edges of the two horizontal steel beams sliding on rollers.

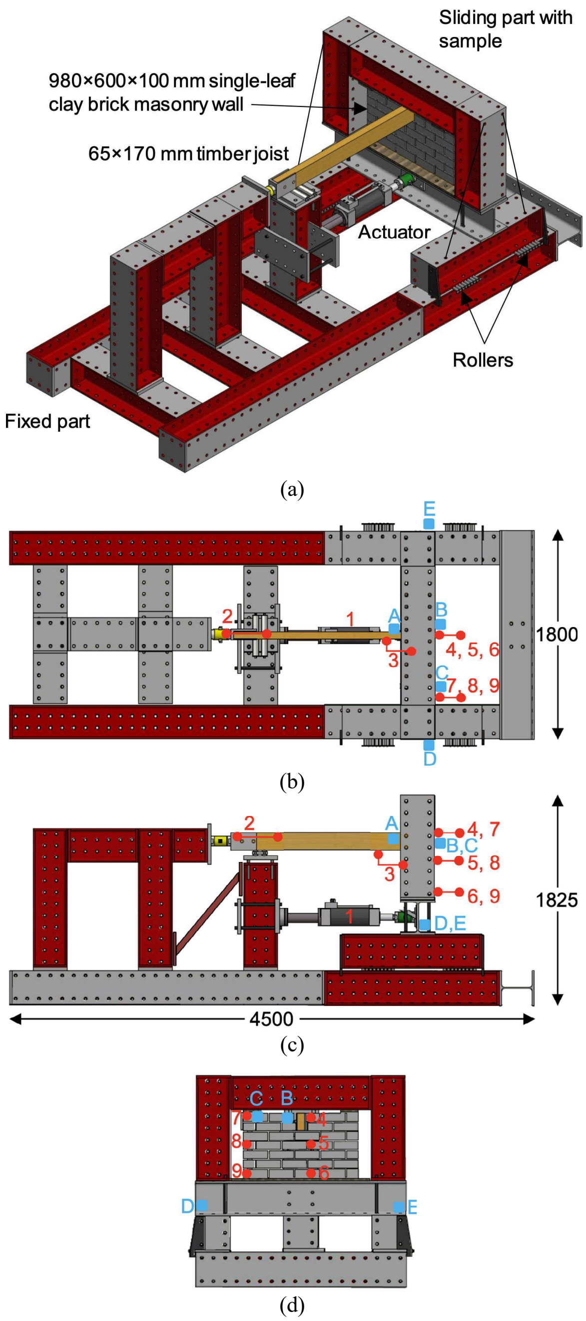


Figure 3: *Test setup and measurement plan: 3D view (a), plan view (b), side view (c), front view (d); dimensions in mm*

A weight of 100 kg was hanged at mid span on the joist, simulating the loads on the portion of floor around the single connection in practice (0.5 kN). No further precompression was applied to the wall, because the tested configurations represented connections at roof level, as already stated in section 2.2.

The monotonic, cyclic and dynamic tests were carried out under displacement control, and the force developed by the connection was recorded by a load cell behind the timber beam. The displacements were not imposed directly to the joist, but more realistically to the bottom part of the wall. In this way, it was possible to use a single setup for all three test typologies.

Figures 3b to 3d show the adopted measurement plan: for monotonic and quasi-static tests, potentiometers were used (sensors 1-9); only for high frequency dynamic tests, accelerometers were also placed (sensors A-E, with maximum capacity 2.5g), to more accurately detect the connection's response due to a sudden solicitation. Sensor 3 was the most meaningful source of information, because it measured the relative displacement between joist and wall: all graphs reported in section 3 refer to it. For strengthened configurations a larger number of sensors was adopted, to detect also local mechanisms, such as deformation of steel angle or brackets, screws' displacement, and sliding between timber elements.

Besides these sensors, for configurations E and F, for which very limited relative displacement and more brittle failures were expected, digital image correlation (DIC) technique was used, to have a complete coverage of the samples' response. The adopted DIC system consisted of two cameras, having a resolution of 4096×3000 pixels each. This allowed the detection of out-of-plane and three-dimensional mechanisms, also for dynamic tests. A random pattern with matt black colour was applied to the front side of the wall (Figure 4).

With regard to the adopted sign convention, in the graphs of section 3 a positive displacement corresponds to a pulling force on the connection as recorded by the load cell, a negative one to a pushing action.

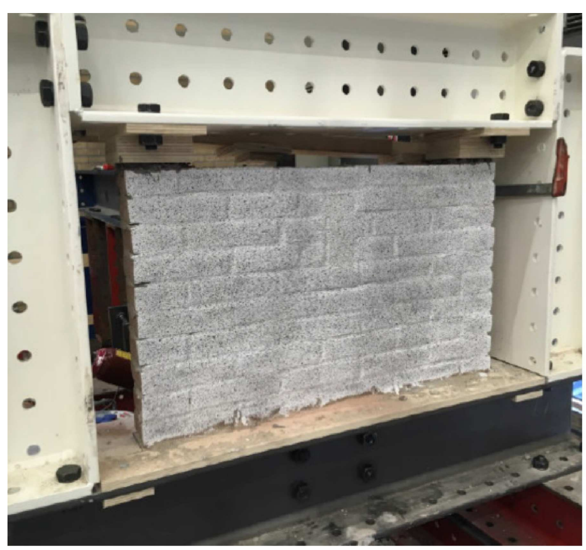


Figure 4: *Pattern applied to the wall for digital image correlation technique*

3 RESULTS

3.1 MONOTONIC TESTS

The configurations showed a very different response in terms of strength, stiffness and ductility: the retrofitting measures were chosen with the purpose to compare solutions with high strength and ductility (C, D, G), and more brittle options, but with great resistance and stiffness (E, F). A first overview of the connections' responses is given in Figure 5, showing the results of monotonic tests. The recorded behaviour determined the displacements' amplitude of quasi-static reversed-cyclic tests, and was in general confirmed by them.

As can be noticed, as-built configuration A displayed a frictional response with almost no capacity of load transfer between joist and wall. Joint type B, due to the shape of the hook anchor, was able to involve part of the wall around the connection, reaching a much higher pulling strength. Retrofitting option C was characterized by yielding and bending of screws and steel angle, with pull-out failure of one of the two anchors at large displacements. Configuration D showed a very ductile behaviour, due to yielding of screws and steel brackets. Joint type E exhibited high strength and stiffness, but with a brittle failure caused by the detachment of the hook anchor from the incision in which it was embedded; it should be noticed that the epoxy layer did not break, but the failure was caused by cracking of bricks and mortar around it. Connection type F displayed a stiff response as well, with high strength; ductility was limited and in general related only to cracking in the masonry around the screws. Finally, configuration G showed the best balance among strength, stiffness and ductility, especially due to the yielding of nails and screws and the good anchoring to the masonry.

3.2 QUASI-STATIC REVERSED-CYCLIC TESTS

Cyclic tests confirmed the response of the connections observed for monotonic tests. For the purpose of comparison, the graphs of Figures 6-12 are all reported at the same scale. Configuration A (Figure 6) showed again a purely frictional behaviour and a very low force transfer between joist and wall; no signs of failure were detected. It was observed that the strength might be increased when the joist is slightly tilted in the mortar pocket, as it happened for sample A-QS-3.

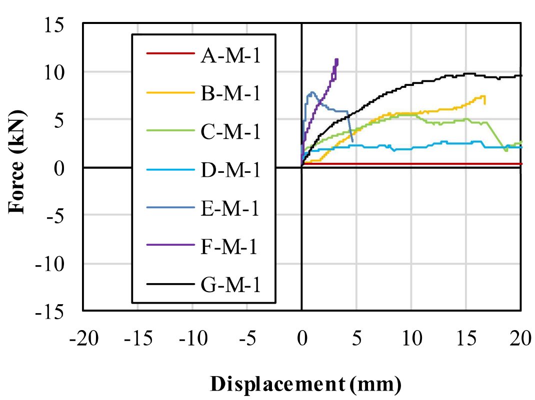
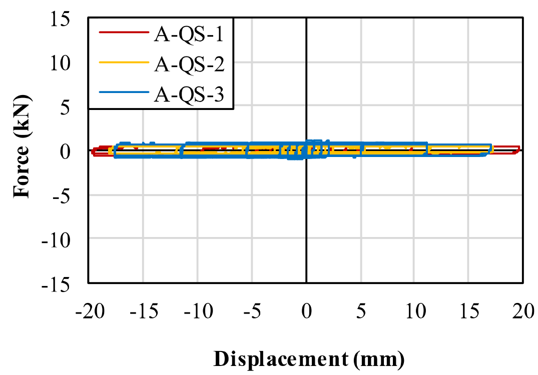


Figure 5: Monotonic responses of configurations A-G

Connection type B (Figure 7) showed a frictional behaviour as well when pushing the connection, since the hook anchor was free to move. Instead, in the opposite loading direction, the wall was also involved in the resisting process, as it happened during the monotonic test, due to the presence of the vertical part of the anchor itself. Although this configuration was an as-built one, its capacity interestingly appeared to be adequate in terms of strength, even if reached only in one of the two loading directions. Sample B-QS-3, unlike the other two, showed a more symmetric response: this occurred because after the first initial cycles with frictional behaviour in the pushing direction, a mortar particle detached from the masonry pocket, remaining clamped between the anchor and the wall, and causing an increase in the transferred load. The failure was caused by the (limited) cracking of the masonry wall around the hook anchor.

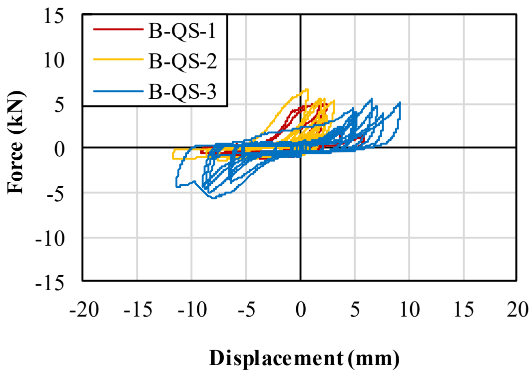
Retrofitting option C appeared to be a good compromise between strength and ductility. In the initial phases a quite uniform behaviour among the three tested configurations was obtained. The ultimate strength was increased by more than ten times compared to configuration A. Like for the monotonic test, moderate yielding of the screws was observed; for large displacements, bricks' or mechanical anchors' extraction occurred from the wall, which displayed many cracks after testing.



(a)

(b)

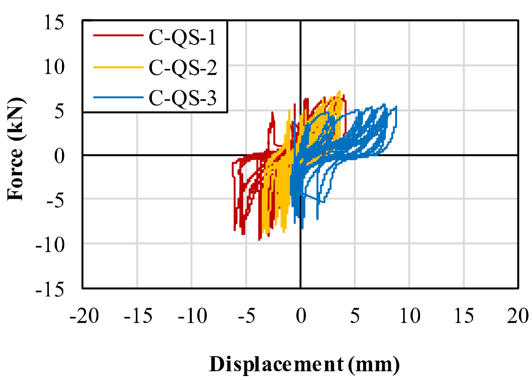
Figure 6: Quasi-static cyclic response of configuration A (a) and joint after testing with no signs of failure (b)



(a)

(b)

Figure 7: *Quasi-static cyclic response of configuration B (a) and joint after testing with limited cracking on the wall (b)*



(a)

(b)

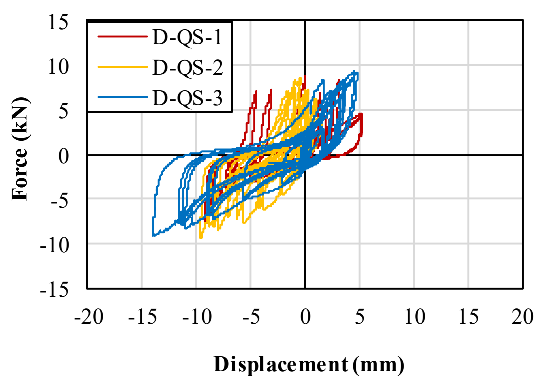
Figure 8: *Quasi-static cyclic response of configuration C (a) and joint after testing (b)*

Configuration D displayed moderate values of strength and stiffness, with high ductility due to bending and yielding of screws and steel brackets. Like joint type C, for large displacements cracks on the wall occurred, leading to a non-symmetric behaviour (Figure 9).

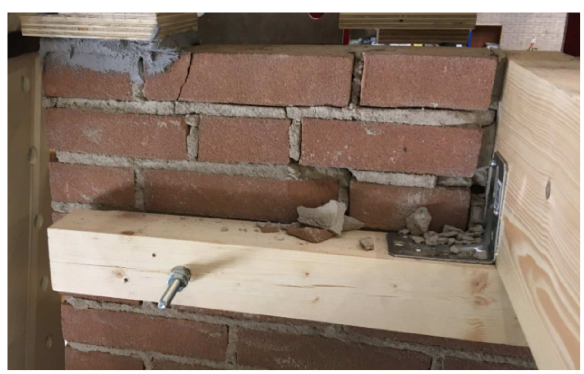
Configuration E was developed in order to obtain a very stiff and resistant connection, although renouncing to ductility. This objective was appropriately reached, as can be noticed from Figure 10. The main failure mode was the detachment from the wall of the glued part of the hook anchor, together with limited yielding of the nails connecting it to the joist. Like it was observed during the monotonic test, the failure was not related to the epoxy, but to the cracking of bricks and mortar around the incision.

Configuration F was designed with the same purpose as option E. In this case, even more strength was retrieved due to the efficient load transfer between joist and wall, and the three specimens exhibited in general a very similar response (Figure 11). No signs of failure were observed in the screws and the timber joist, but a large and distributed crack pattern was visible on the walls.

Configuration G, designed to be a dissipative option, showed a moderate capacity, linked to high ductility and energy dissipation, caused by yielding and bending of screws and nails and, for large displacements, also by cracking in the walls. Furthermore, the three tested samples displayed a very similar global response, as shown in Figure 12.

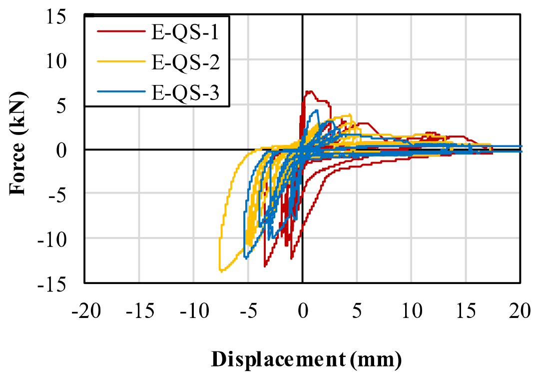


(a)



(b)

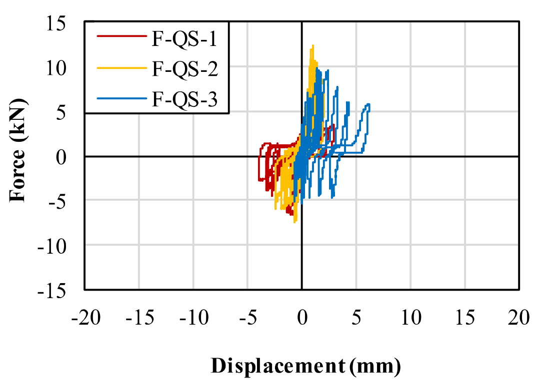
Figure 9: *Quasi-static cyclic response of configuration D (a) and joint after testing (b)*



(a)

(b)

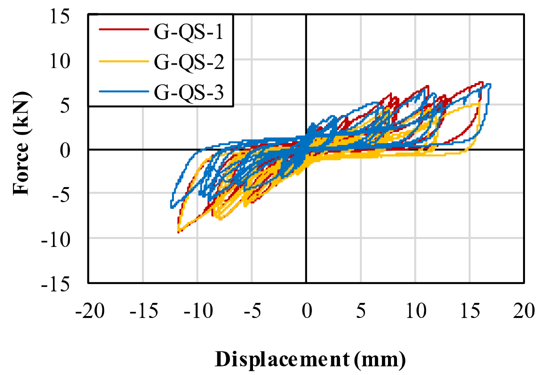
Figure 10: *Quasi-static cyclic response of configuration E (a) and joint after testing (b)*



(a)

(b)

Figure 11: *Quasi-static cyclic response of configuration F (a) and joint after testing (b)*



(a)

(b)

Figure 12: *Quasi-static cyclic response of configuration G (a) and joint after testing (b)*

3.3 HIGH-FREQUENCY DYNAMIC TESTS

The responses of the specimens loaded with the chosen dynamic signal were similar to the quasi-static cyclic ones. However, especially in the pushing loading direction an impact effect was noticed, increasing the capacity of the strengthened joints. For the pulling direction, the play present in the joint was also more evident.

Configuration A (Figure 13) displayed again a frictional response, with slightly higher forces compared to the quasi-static case.

Configuration B (Figure 14) showed a frictional behaviour when pushing the anchor, and an increase in strength when pulling it. This increase occurred in a slower way compared to the quasi-static cyclic response, probably because of the higher play in the connection induced by sudden loading.

Configuration C (Figure 15) exhibited an equivalent response to quasi-static behaviour. However, for the pushing direction, higher peak forces were reached due to the aforementioned impact effect.

For configuration D (Figure 16) a slightly more flexible and less resistant behaviour was observed when pulling the joint, compared to the quasi-static case.

Configuration E (Figure 17) showed higher peak forces in the pushing direction due to the impact effect, while the overall response was equivalent to the one observed in quasi-static tests.
Configuration F (Figure 18) displayed larger force transfer in pushing as well; therefore, under dynamic loading this connection type became nearly symmetric in terms of peak strength and stiffness.
Configuration G (Figure 19) exhibited the same cycles' shape as the one in quasi-static tests, but with slightly higher loads in both directions. Ductility and energy dissipation were therefore again present, with an enhanced transfer of force in the connection, probably again due to the impact effect.

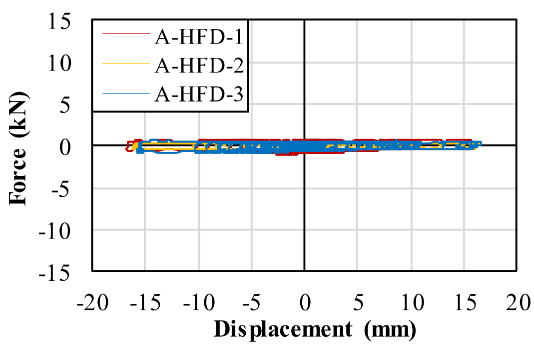


Figure 13: *Dynamic response of configuration A*

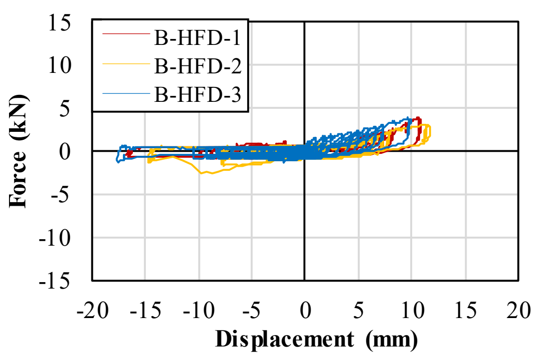


Figure 14: *Dynamic response of configuration B*

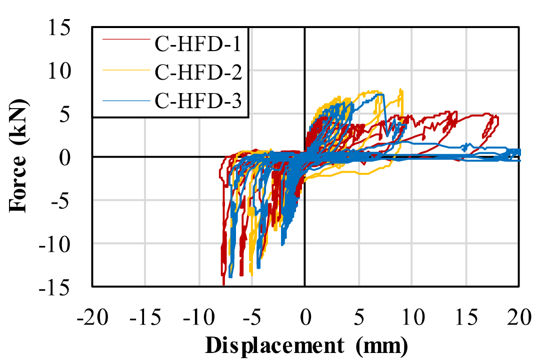


Figure 15: *Dynamic response of configuration C*

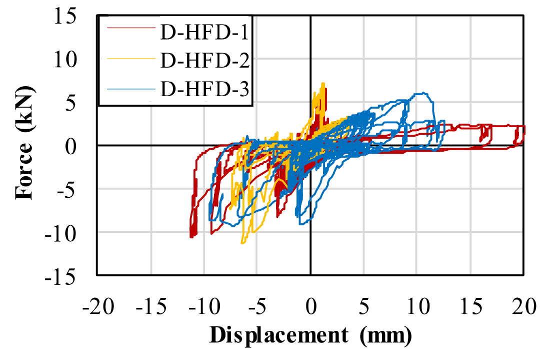


Figure 16: *Dynamic response of configuration D*

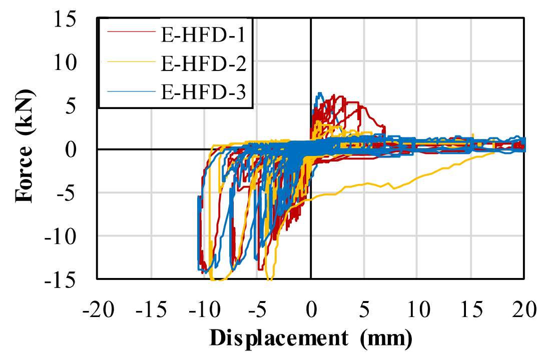


Figure 17: *Dynamic response of configuration E*

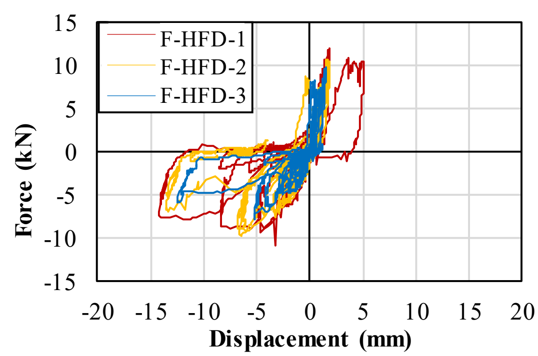


Figure 18: *Dynamic response of configuration F*

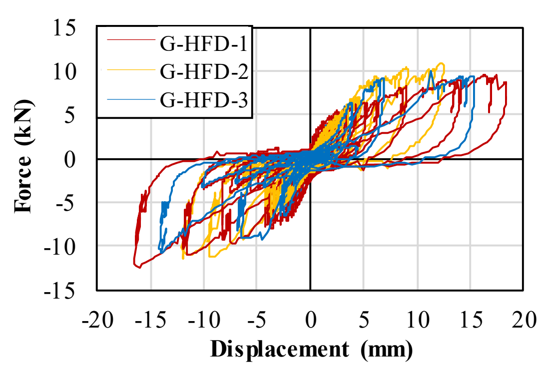


Figure 19: *Dynamic response of configuration G*

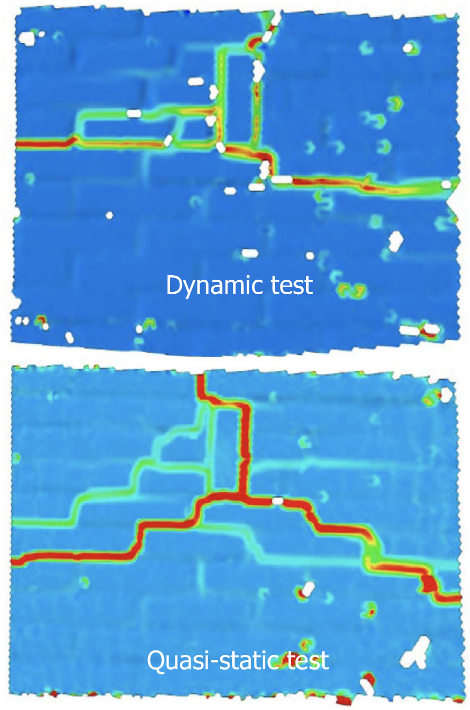
4 DISCUSSION

According to test results, all strengthening measures showed moderate or high capacity against horizontal loads, and reference values of strength and stiffness were quantified for them as input for structural calculations. Configuration B, even if non-strengthened, was also capable of sufficient load transfer, although only when pulling the joint. In general, the observed variation in strength, stiffness, and ductility when performing monotonic tests, was confirmed by quasi-static tests. It is, instead, of relevance to compare quasi-static tests to dynamic tests.

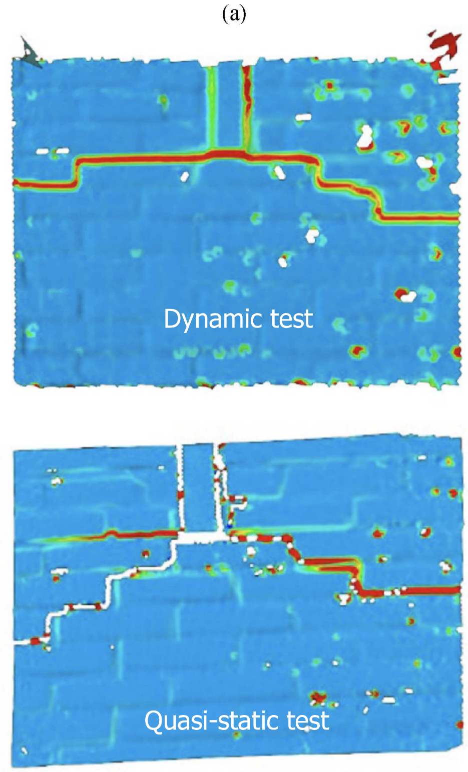
As a first, general remark, quasi-static tests appeared to be reliable and conservative, and constituted a good estimation of the possible seismic behaviour of each connection. Yet, they were not able to capture specific features of dynamic loading, such as the observed impact effect or the higher play induced in the joint's components. Nevertheless, both strength, stiffness and global behaviour under sudden seismic forces were well represented by quasi-static cyclic tests.

Another interesting aspect was related to the overall damage on the samples at the end of the tests, which appeared to be lighter after dynamic loading with respect to quasi-static one. This could be explained by considering the specific adopted signal, representing a shallow, human-induced and light earthquake, with a sudden movement at the beginning and a rapid decrease in amplitude. In other words, in quasi-static tests the number of cycles to which a sample was subjected, was much larger compared to dynamic tests. This reflected on the observed damage on the connections and the walls: with seismic signals having longer duration and containing more cycles, a similar or larger damage could be expected for samples subjected to dynamic loading.

The presence of less damage on specimens tested with a real seismic signal was confirmed by DIC results as well: at the displacement corresponding to peak force, a deeper and more extended crack pattern is noticeable on the walls for samples tested under quasi-static cyclic loading, as shown in Figure 20: a red colour indicates an almost complete crack opening, a blue one an intact material; white colour in a crack corresponds to a total detachment of mortar, as is recognizable in Figure 20b.



(a)



(b)

Figure 20: *Comparison between the crack pattern observed at the displacement corresponding to peak force for dynamic and quasi-static tests in configuration E (a) and F (b)*

5 CONCLUSIONS

In this work, an experimental campaign to characterize the seismic response of as-built and strengthened timber-masonry connections was presented. Their strength, stiffness and energy dissipation were evaluated through monotonic, quasi-static reversed-cyclic, and high-frequency dynamic tests.

As-built configurations, in particular the masonry pocket, showed low strength and stiffness, while a considerable improvement in the response was achieved with the proposed strengthening techniques; for the most ductile ones, energy dissipation was relevant as well.

A comparison among the pushing and pulling peak strengths of all tested joints is provided in Figure 21. From this overview, the following aspects are briefly recalled as concluding remarks:

- An evident improvement compared to as-built configurations was achieved with all strengthening measures: for both loading directions, if compared to joint type A; in pushing with respect to connection B.

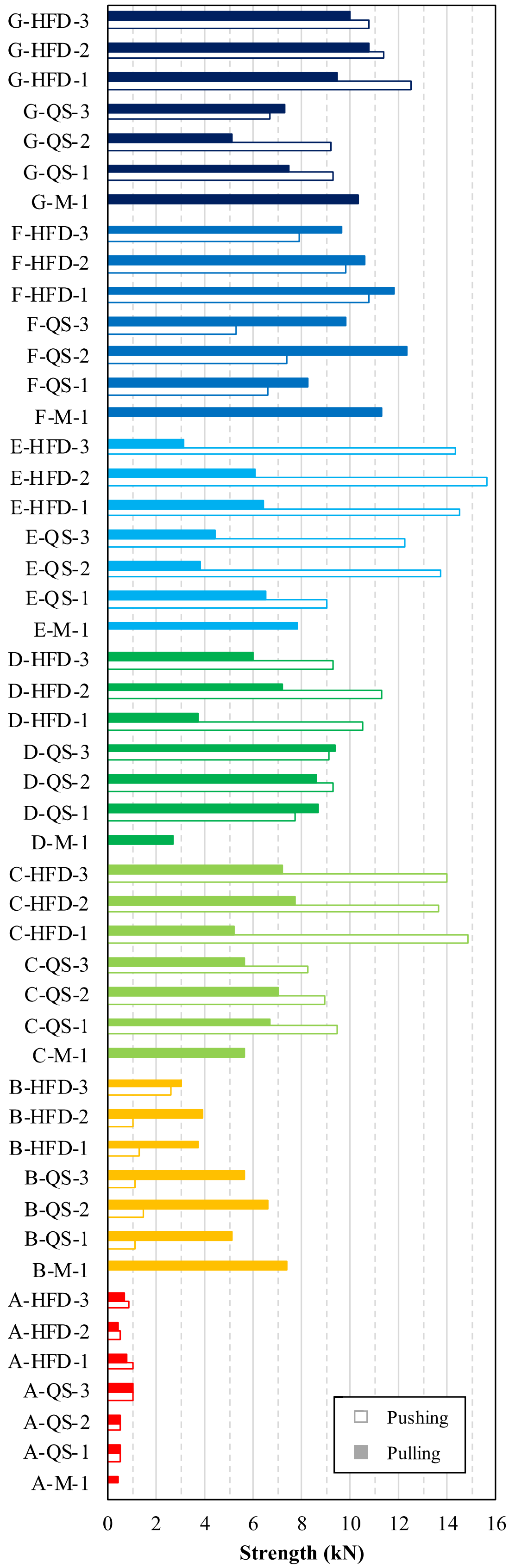


Figure 21: *Comparison among the recorded peak strengths in pushing and pulling for the tested configurations: the colours help distinguish each one of them*

- Considering the intrinsic inhomogeneity of both timber and masonry, the dispersion in the result was acceptable, and reasonable uniformity was especially observed for joints C, F and G.
- The pushing response under dynamic loading was characterized by a higher strength due to the impact effect related to the sudden force transfer.
- When subjected to a typical short-duration human-induced earthquake signal, the connections experienced less damage compared to their response under quasi-static cyclic loading.

Further research is ongoing to characterize each retrofitting solution from the analytical point of view, with the aim of generalizing the results for other contexts as well.

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