Strengthening of masonry wall load bearing structures with reinforced plastering mortar solution

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SUMMARY:
One of the most common seismic strengthening techniques for load bearing masonry walls, here referred to as *reinforced plastering mortar* solution, consist in the addition of outer leafs (preferably on both faces of existing walls) made of premixed structural mortar or sprayed concrete, reinforced with strengthening meshes (steel or fibreglass).

This paper presents on the results of a sequence of experimental testing stages devised to determine the strengthening effects and to identify the most effective detailing procedures for this solution. The initial testing stage was focused on the behaviour of composite mortar-mesh specimens, subjected to tensile tests. In a later stage, a group of eight nearly full scale masonry wall models, unreinforced and reinforced with steel or fibreglass meshes, were subjected to in-plane and out-of-plane imposed displacements. The details of the implementation of this strengthening solution in a school in the Algarve are also reviewed.

Keywords: reinforced plastering mortar, masonry walls, experimental tests, in-plane, out-of-plane

1. INTRODUCTION

A significant part of the building stock in European cities is made of unreinforced masonry wall load bearing structures, known to present an increased vulnerability (Kaplan et al, 2010) when subjected to strong motion earthquakes. This fact explains the need for progressive large scale seismic strengthening with techniques accessible to ordinary construction contractors.

One of the most common strengthening techniques, here referred to as *reinforced plastering mortar* solution, consist in the making up of outer leafs (also known as outer layers), in existing walls, preferably on both faces, in which these leafs are made of premixed structural mortar or sprayed concrete (shotcrete), reinforced with properly anchored strengthening meshes (steel or fibreglass meshes are the most common). Figure 1 illustrates one possible solution, both for interior (brick masonry) and exterior walls (rubble stone masonry).

In spite of the fact that this solution has been applied in some European earthquake prone regions, Gigante (1998), Penazzi et al (2011), there is still some lack of guidance, both in terms of design (or assessment) as in terms of the most effective detailing rules. Two of these rare studies were provided by ElGawady et al (2004) and Costa et al (2010a, 2010b). The advantages of this solution, aimed at the improvement of both in-plane and out-of-plane behaviour of the walls, is thought to greatly depend on some detailing issues, such as the anchorage of the reinforcing mesh at the extremities (or between floors, when interrupted by these) and the across-wall clamping of the reinforcement leafs (preventing the separation between the slender reinforcement leafs and the pre-existent wall). The connection to reinforced concrete floors (when these are present) may also prove critical in taking advantage of the beneficial diaphragm effect.

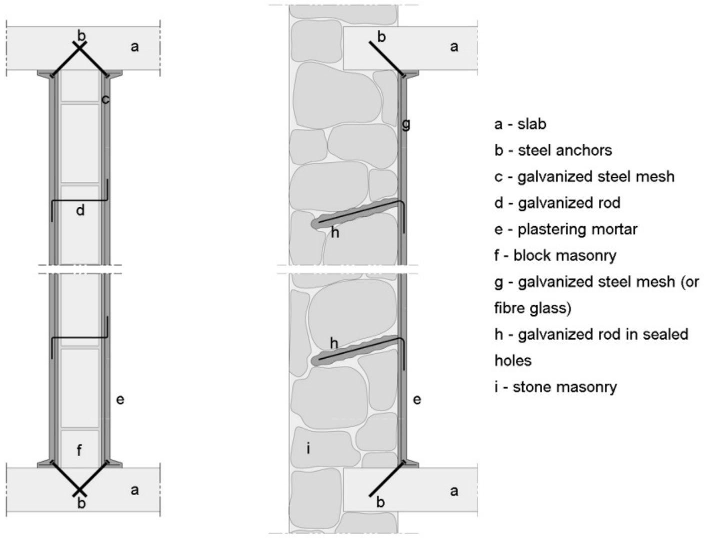

Figure 1. Typical details for steel mesh reinforced plastering mortar solution

This paper focuses on the results of a sequence of experimental testing stages devised to determine the strengthening effects and to identify the most effective detailing procedures for this solution, namely for the different variants of the solution adopted for the strengthening of a school building of the Tomás Cabreira secondary school, in the Algarve (southern Portugal).

In the initial testing stages, some composite mortar-steel mesh specimens were subjected to tensile tests to ascertain the beneficial effect of the inclusion of the reinforcement and to identify the most common composite failure modes.

In later stages, a group of eight nearly full scale masonry wall models, unreinforced and reinforced with steel or fibreglass meshes, were subjected to in-plane and out-of-plane imposed displacements (in addition to constant vertical loads, simulating upper storeys' weight). This latter experimental campaign is reviewed, with an emphasis on the collapse modes (shear/sliding/rocking for in-plane and crushing/cracking for out-plane) and the influence of detailing measures.
Some of the details of the strengthening works carried out in the Tomás Cabreira secondary school are presented in the end.

2. TESTS ON COMPOSITE MORTAR-STEEL MESH SPECIMENS

The outer layer made by the structural mortar (or sprayed concrete) and mesh may be considered a composite material, subjected to predominately tensile forces particularly when the strengthened walls are subjected to out-of-plane forces (or displacements). The behaviour of this composite material was studied through a series of tensile tests on specimens replicating the outer layers of the strengthening solution adopted for interior faces of the strengthened walls in the previously referred school. These tests were conducted on LERM's (Laboratory of Structures and Strength of Materials, IST, Lisbon) ±250kN / ±50mm universal testing machine. In this case the reinforcement consisted in expanded steel wire mesh and the matrix consisted in cement-based structural mortar (nominal compression strength over 6MPa). The strengthening solution adopted for the exterior faces of exterior walls (with fibreglass mesh) was not tested in this stage.

Initially, some tensile tests were carried out with specimens made only by the reinforcement, considering two different testing directions, since the expanded steel wire mesh is anisotropic. Afterwards, the tests were conducted with nearly full-scale composite specimens (4cm thickness, see figure 2 right and figure 4).

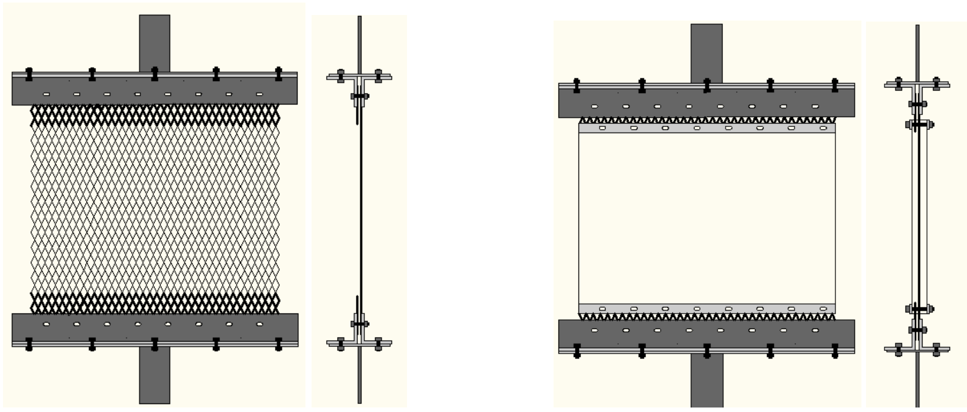

Figure 2. Reinforcement-only (left) and composite (right) specimens and testing device

The first tests (steel mesh-only) have clearly shown that the strength of the expanded wire mesh is extremely different in the two main directions. The tested mesh presented a lozenge shape, in which the strength was much higher when subjected to forces along the larger dimension. For 650mmX650mm specimens, the average tensile strength was of 24.1kN (see figure 3) or less than 1kN, respectively along the larger and shorter dimensions of the lozenges. Specimens tested along the larger dimension of the lozenge shape presented clear tensile fracture of the wires, whereas specimens tested along the shorter dimension did not present fracture (the displacement increased indefinitely along the range of the testing machine, without any sign of tensile fracture and extremely low values of force).

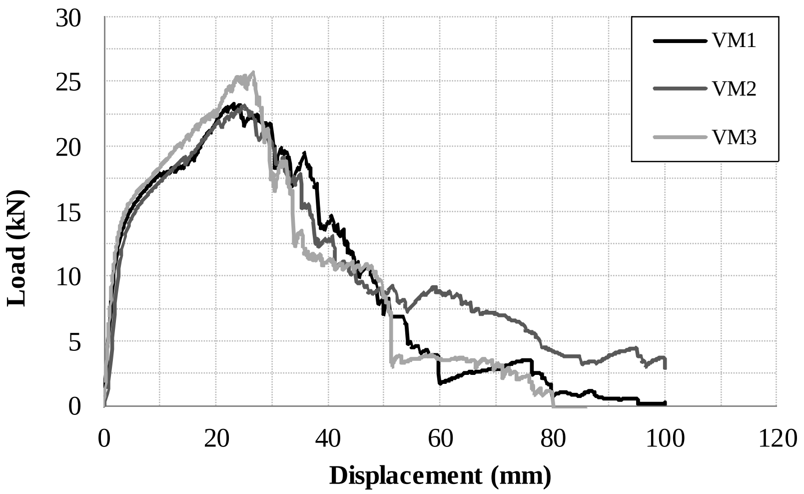

Figure 3. Tensile force versus displacement chart for material (steel mesh) specimens tested along the larger dimension of the lozenge shapes

The composite test specimens were similar in shape to those of the steel mesh only, with a total thickness of 40mm. Apart from the general displacement and force measurements (from the testing machine) each of these specimens was further instrumented with strain gauges (one in each face) and a displacement transducer (to measure the total elongation within the specimen itself, therefore not affected by slippage and deformation of the testing device). The first of these tests (tension along the larger dimension of the embedded lozenge steel wire mesh) presented a series of premature failure modes, generally related to failure at the specimen clamping systems (existent at the extremities).

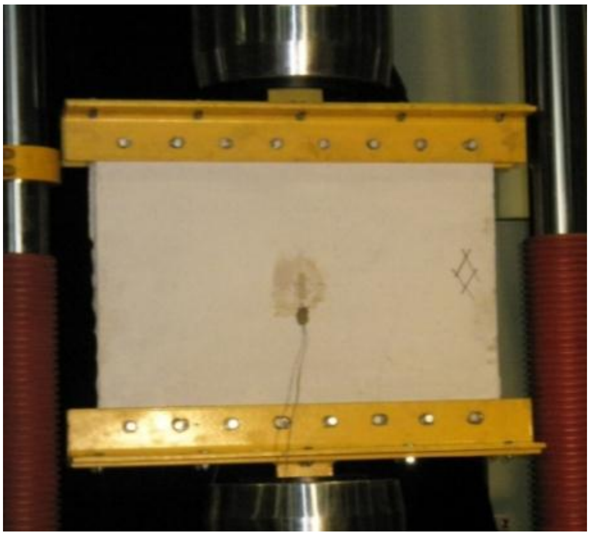

Figure 4. Composite mortar-steel mesh specimen test

The first series of composite tests was performed for three specimens subjected to tension along the larger dimension of the lozenge shapes. In two of these tests (maximum force of 40.2 and 15.4 kN) there were indications that tensile fracture occurred prematurely, due to effects induced at the extremities (clamping system limitations). The 3rd tensile test of these specimens led to a maximum force of 40.1 kN. Considering the results of the material tests, only one specimen was manufactured for tension along the shorter dimension of the lozenge shapes. This sole specimen sustained a maximum force of 6.5 kN and tensile collapse occurred with a combination of wire fracture and mortar cracking (as expected).

The main conclusion of the composite mortar-steel mesh tensile tests is that the reinforcement (expanded steel wire mesh) presents a clear anisotropy, indicating that the steel mesh should be placed with the larger dimension of the lozenge shapes along the vertical direction (direction along which occur the tensile forces of the reinforcement for walls subjected to out-of plane forces/displacements). Another conclusion, limited by the reduced number of composite specimens successfully tested, is that the mortar-steel mesh behaves as a composite material, in which the tensile collapse occurs for a combination of wire fracture and mortar cracking.

3. TESTS ON MASONRY WALL MODELS

The second series of tests was performed on a group of eight nearly full scale masonry wall models, replicating the different strengthening solutions and detailing to be applied in a school in the Algarve (southern Portugal). Of these, four specimens were subjected to in-plane cyclic forces and the remaining four to out-of plane forces cyclic forces, making up a group of four pairs of otherwise identical specimens. Apart from the in-plane and out-of-plane cyclic forces, all specimens were subjected to constant vertical loads, simulating the dead and live load effects on load bearing masonry walls.

Each group of four specimens was composed of an unreinforced (UMW) and three reinforced (SR1MW, SR2MW and GRMW) specimens. SR1MW and SR2MW corresponded to reinforced interior walls with expanded steel wire mesh reinforcement (SR) and detailing prescribed at the ground level (SR1, tying of the mesh though steel angles anchored in the wall foundation) or to the detailing prescribed in intermediate floors (SR2, tying of the mesh through 8mm steel rebars spaced at 70cm, crossing the intermediate floor slabs, made of reinforced concrete). Models GRMW corresponded to exterior walls, reinforced in the external face with fibreglass mesh (GR) and in the internal face similarly to SR1 specimens.

The remaining detailing aspects – overlap of reinforcing mesh, across wall clamping of composite reinforcement layers – followed the indications set forth in the strengthening works of the school.

Due to the diversity of solutions existent in the school – rubble masonry, mostly for exterior walls, and brick masonry for the interior walls – the tested wall specimens were made of hollow mortar blocks presenting strength and deformation characteristics similar to those of low to medium quality workmanship masonry walls existent in southern and central Portugal (compressive strength of 3.25 MPa).

The general dimensions of the wall specimens were of 1m (length), 1m (height) and 0.23m (thickness), as shown in figure 5. The proportions of these specimens tried to comply with those of the most common load bearing masonry piers existing in the school, so that the prevailing in-plane failure modes (Magenes and Calvi, 1997) could be respected.

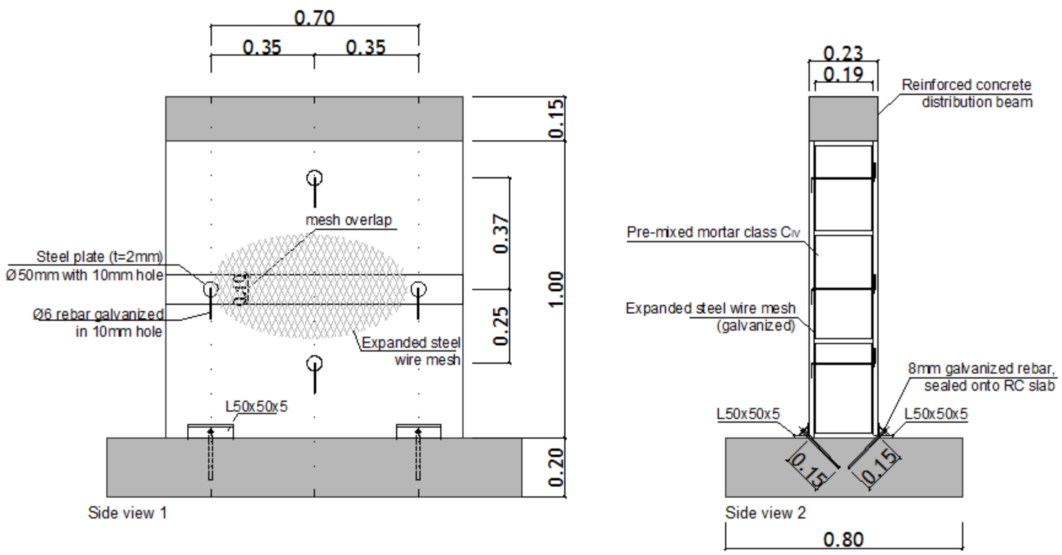

Figure 5. Specimen dimensions and detailing (example, SRW1, dimensions in m)

Tests were carried at the Instituto Superior de Engenharia, Universidade do Algarve, Faro, Portugal, close to the work site of the Tomás Cabreira secondary school.

The experimental setup was similar for in-plane and out-of-plane tests. The testing frame allowed for the accommodation of the specimens in both testing directions, with only slight variations (shown in figure 6).

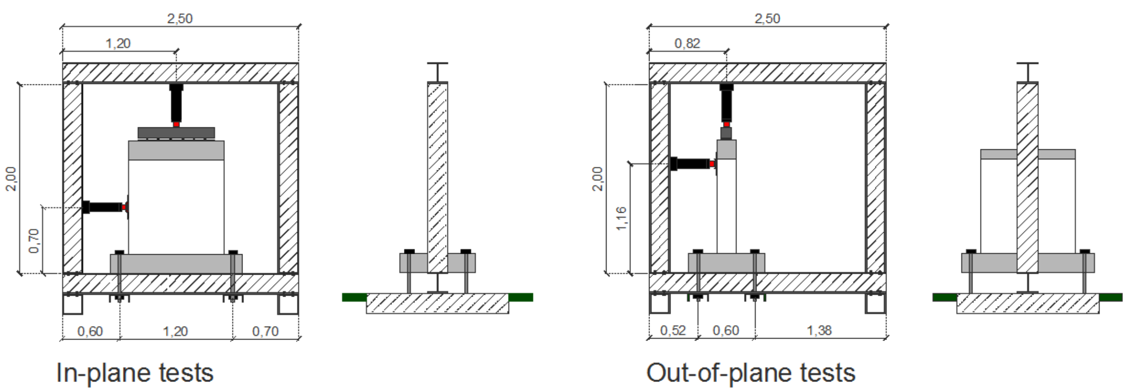

Figure 6. In-plane and out-of-plane experimental setup (dimensions in m)

The instrumentation (not shown in figure 6) consisted in two load cells (vertical and horizontal jacks) and a series of up to six displacement transducers to measure the rigid-body motion of the model and the internal deformation at different heights. The vertical load, applied initially and maintained al throughout the tests corresponded to a vertical stress of 0.25 MPa, considered representative of the vertical load effects on load bearing masonry walls.

The results of the out-of-plane and in-plane tests are presented separately in sub-sections 3.1 and 3.2, respectively.

3.1. Out-of-plane tests
In specimen UMW, collapse was initiated by a single discrete crack developing at the base of the wall. The maximum horizontal load was of 11.63kN, corresponding to a strength plateau initiated at displacement values of 1mm and prolonged to 6.2 mm.

Specimen GRMW was tested in such a way that the fibreglass mesh strengthening layer was predominantly subjected to tension (compression on the expanded wire mesh layer). Collapse was apparently due to tensile failure of the fibreglass mesh, leading to the development of a crack just above the anchoring detail. The maximum force and corresponding displacement were of 14.45 kN and 10.42 mm, respectively.

Specimens SR1MW and SR2MW presented a similar collapse mode due to failure of the anchorage at the specimens' base. The ultimate loads (and displacements at failure) were, respectively, of 17.2 kN (17.23mm) and 33.1 kN (33.13 mm). Generally speaking both these specimens presented higher deformability (and deformation capacity) than the unreinforced model. The force-displacement charts, show in figure 7, seem to indicate some anomalous behaviour in specimen SR1MW.

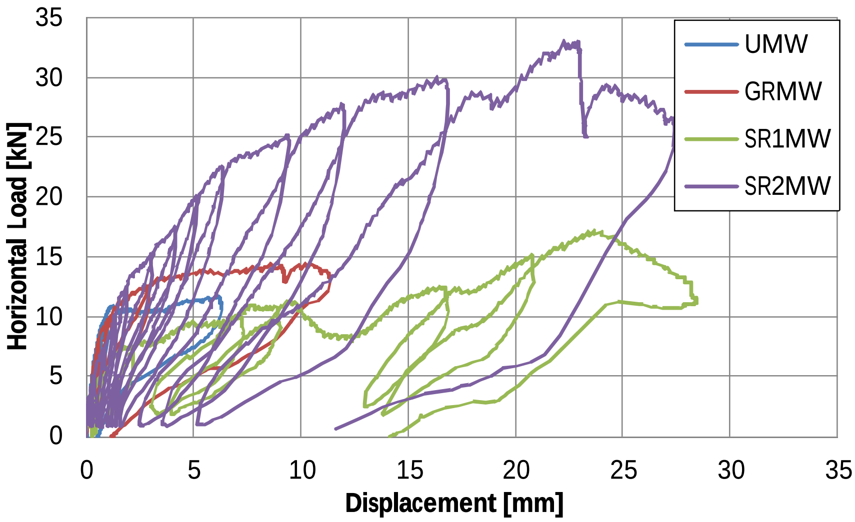

Figure 7. Force-displacement chart for all out-of-plane tested specimens

Apart from the strength (and deformation) improvements for the strengthened specimens, another important indication lies in the accumulated dissipated energy at maximum force, computed as the work performed by the horizontal force up till then. The corresponding values were of 66.4 kNm, 140.8 kNm, 312.2 kNm and 692.7 kN for specimens UMW, GRMW, SR1MW, SR2MW and in that order.

3.2. In-plane tests

The UMW specimen collapsed by shearing through the opening of the two diagonal cracks. The specimen presented a maximum shear strength of 80.47 kN with an associated horizontal displacement of 0.95 mm.

Similarly to the UMW specimen, the GRMW specimen collapse by shearing through the opening of the diagonal crack on the glass reinforced surface. It occurred for a horizontal load of 160.76 kN and a displacement of 2.40 mm.

Unlike the previous specimens, SR1MW presented a sliding collapse at the bottom of the wall at a 145.63 kN horizontal load with the corresponding displacement of 12.41 mm.

For the SR2MW specimen the fracture initiated at the load application point for a load of 151.84 kN. The horizontal displacement at this stage was 2.62 mm. The force-displacement relations are drawn in figure 8.

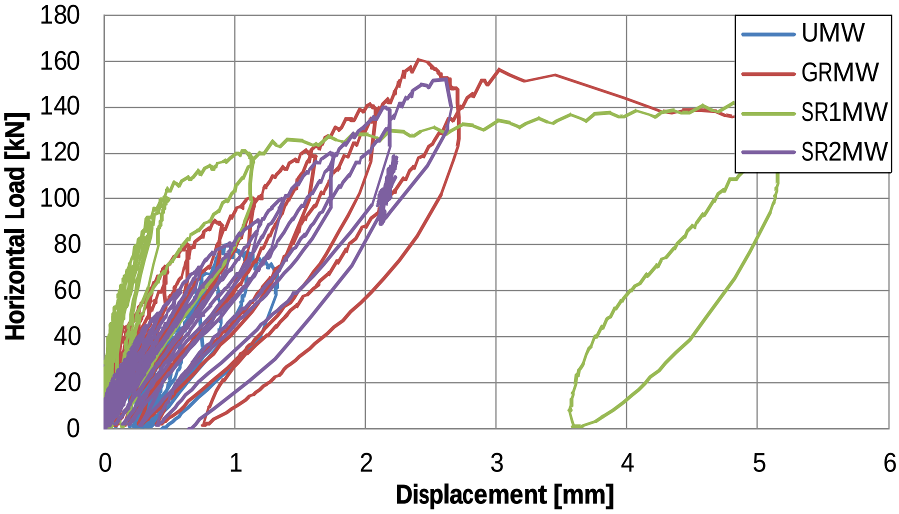

Figure 8. Force-displacement chart for all in-plane tested specimens

Ignoring the obviously important improvements to the strength and deformation performances, the reinforced specimens also presented a greater accumulated dissipated energy at maximum force, with the values of 358.81 kNmm, 1730.64 kNmm and 325.41 kNmm for GRMW, SR1MW and SR2MW respectively, in relation to the 82.77 kNmm obtained for the UMW specimen.

4. CASE STUDY: TOMÁS CABREIRA SECONDARY SCHOOL (FARO, ALGARVE)

As previously referred to in this paper, the experimental tests were performed to assess the benefits of strengthening the load bearing walls of the Tomás Cabreira secondary school (located in the Algarve, southern part of mainland Portugal, an area of increased seismic hazard). These tests were also intended to validate the worthiness of the detailing measures (anchorage of the meshes at the base, transmission of forces between floors and clamping arrangements).

The main building of this school was originally erected in the beginning of the 20th century but suffered major rehabilitation and remodelling works in the late 1940s, so that the existing structure before the intervention resembled a building from that period: rubble masonry or ceramic brick load bearing masonry walls with reinforced concrete floors (lightened in some areas).

The walls requiring strengthening were identified through a numerical linear finite-element model (figure 9) representing the main building. This numerical model was further calibrated and validated, considering modal identification results (experimentally determined fundamental mode shapes and frequencies).

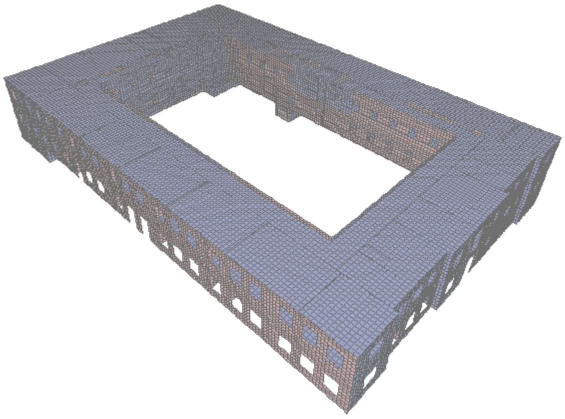

Figure 9. Numerical model of the main building

The safety assessment of the load bearing masonry walls was focused in the in-plane resistance (the intervention would comprise stitching these walls to the floor structures, so that local out-of-plane failure modes could reasonably be discarded).

Strength calculations were performed for each wall element (and globally also) considering a Mohr-Coulomb failure criterion, corresponding to a more realistic sliding-shear failure mode (diagonal cracking and rocking failure modes could be discarded due to the aspect ratio of the walls and levels of vertical loading). For the unreinforced masonry, the mechanical properties (friction angle, ϕ, of 17º and cohesion, C_u, of 0.06 MPa) were based in the Italian code OPCM 3274/3431(2005), considering a compact brick masonry with lime mortar configuration. A Confidence Factor of 1.35 was adopted according to the Knowledge Level reached (LC1) which affects the friction coefficient and the cohesion of the material. Given the difficulties in quantifying the benefits of the reinforced plastering mortar strengthening technique, it was accepted that it would lead to an increase of the value of the cohesion. The cohesion used for the strengthened masonry (C_u=0.25 MPa) represents a higher quality masonry, given the range of values prescribed in Eurocode 6.

Durability and conservation concerns dictated that fibreglass meshes (alkali-resistant) were applied in the exterior leafs (simulated in the tests by specimen GRMW), whereas galvanised expanded steel wire meshes were used on the interior leafs (replicated by specimens SRMW).

The meshes are nailed (clamped) onto existing walls by means of 6mm steel rebars and donut-shaped steel plates, arranged in quincunx pattern , and anchored with grout within these walls. The rebars are anchored with a 20º inclination or crossing through the wall in case of rubble stone masonry or brick masonry respectively. The donut-shaped steel plates were devised to ease the construction stages, particularly when tying the meshes close to the support.

Steel rebars (8mm) were used to guarantee the continuity of the tensile forces on the strengthening meshes between across floors (between consecutive storeys). The spacing of these continuity rebars was designed taking into account the full tensile resistance of the steel meshes.

On the top (roof ceiling) and base (foundation) a different anchor solution was considered. On the ceilings it was considered a CAE 50x5mm profile chemical anchored smashing the mesh against the masonry.

On the base, two different solutions were considered: one, applied in interior leafs, and other to exterior leafs (elevation leafs). On the interior leafs the CAE profile was also considered on the joint between the ground slab and the wall; on the exterior leafs 8mm steel rebars were used, anchored on the foundation.

A cement-based structural mortar was considered with a nominal compression strength over 10MPa, similar to the one used in the manufacture of the specimens. Figures 10 to 11 depict different stages of the strengthening works.

Figure 10. Steel mesh application on the site (interior leafs)

Figure 11. Fibreglass mesh application on the site (exterior leafs) – South elevation

4. CONCLUSIONS

As referred to previously, the proposed solutions to strengthen the load bearing masonry walls of the Tomás Cabreira School in order to better withstand the seismic actions can be considered effective. The different solutions and detailing measures were shown to significantly improve the mechanical behaviour of the strengthened walls when subjected to in plane or out of plane horizontal loads.

Also worth of note is the improvement to the strength, energy dissipation and deformability is even more significant when the seismic actions are in the plane of the wall.

As such, the proposed solutions meet the established objectives.

AKCNOWLEDGEMENT
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