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Tomislav Kišiček[1], Mislav Stepinac[2], Tvrtko Renić[3], Ivan Hafner[4], Luka Lulić[5]

[1] *Professor*, University of Zagreb Faculty of Civil Engineering, *tomislav.kisicek@grad.unizg.hr*

[2] *Assistant Professor*, University of Zagreb Faculty of Civil Engineering, *mislav.stepinac@grad.unizg.hr*

[3] *Assistant*, University of Zagreb Faculty of Civil Engineering, *trenic@grad.unizg.hr*

[4] *Assistant*, University of Zagreb Faculty of Civil Engineering, *ivan.hafner@grad.unizg.hr*

[5] *Assistant*, University of Zagreb Faculty of Civil Engineering, *luka.lulic@grad.unizg.hr*

Abstract

The earthquake in Zagreb on March 22, 2020 damaged many buildings, most of which were masonry and constructed before the use of earthquake-resistant construction regulations. That quake indicated the need for strengthening masonry buildings for seismic action. Seismic forces are mostly carried by the in-plane shear walls. Therefore, the main problem is proper calculation of the required strengthening of the in-plane shear walls and solving the strengthening details, but not only locally, at the level of individual walls, but also at the level of the whole building, to properly cover all mechanisms of masonry failure and transferring seismic forces to the foundation soil.

The paper presents the most common methods of calculation of in-plane shear wall strengthening with a review of details that are not sufficiently well clarified or are neglected, i.e., poorly used in practice. In addition to the calculation of strengthening of masonry with the reinforced concrete layer (shotcrete) and dowels, modern methods of strengthening are considered, using FRP fabrics or plates and FRCM systems. As these calculation methods are not implemented in the existing codes, various recommendations are used based on similar assumptions, but with different input parameters and safety factors that affect the calculation result and that should be properly interpreted and used in the calculation.

Key words: masonry, shear strengthening, shotcrete, dowels, FRP, FRCM

1   Introduction

The earthquake in Zagreb on March 22, 2020 damaged many buildings, most of which were masonry buildings and constructed before the use of earthquake-resistant construction regulations. The magnitude of the main earthquake was 5.5 according to Richter, followed by another magnitude 5.0 and several smaller ones that occurred in the days after. The epicentre was 7 km from the centre of Zagreb, around Markuševac. The old town (Upper and Lower town) and the suburbs of Čučerje and Markuševec suffered the most.

The buildings in the city centre are typical masonry structures built in the late 19th and early 20th century. They are built of solid brick, masonry is unreinforced, the ceiling structures are wooden beams, and the roof is a wooden structure covered with tiles. Buildings that exceeded its service life, were constructed before seismic regulations and were poorly or inadequately maintained, suffered the most.

That earthquake indicated the need for strengthening masonry buildings for seismic action. In masonry buildings seismic forces are mostly carried by the in-plane shear walls. Proper calculation of the required strengthening of such shear walls is the main challenge. Shear walls should have been considered not only locally, at the level of individual walls, but also at the level of the whole building, to properly cover all mechanisms of masonry failure and transferring seismic forces to the foundation soil. Also, the strengthening details are very important.

2   In-plane strength of unreinforced masonry walls

Unreinforced masonry walls can fail in-plane due to loss of stability as a rigid body, due to sliding, diagonal cracking or toe crushing coupled with horizontal cracking (figure 1). Rigid body instability occurs if the resultant of vertical and horizontal forces falls outside an area of the wall. This type of failure is characteristic of elements with very low vertical pressure or structures without sufficient connection to the horizontal elements. Example of such an element are freestanding chimneys or parts of chimneys above the roof structure.
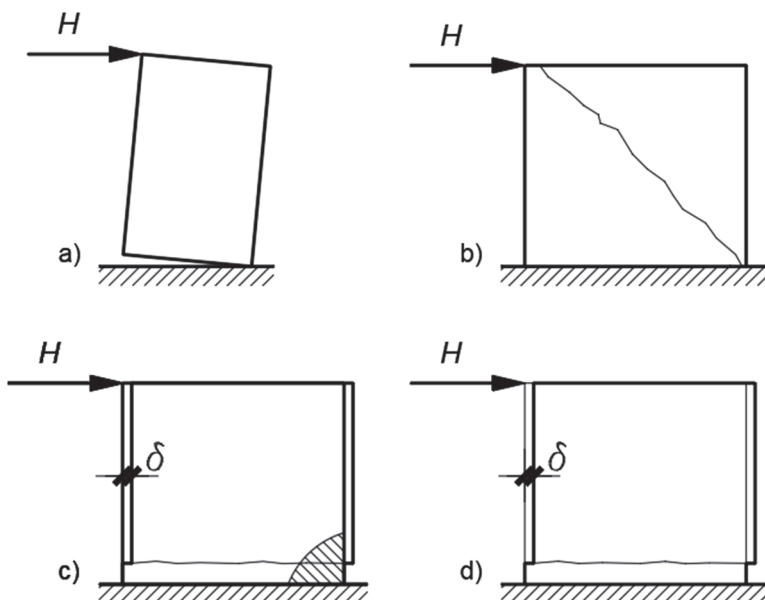
Figure 1. Failure modes: a) loss of stability, b) diagonal failure, c) bending failure, d) sliding failure

Sliding is usually described by a Mohr-Coulomb failure model:

$$f_v = f_{v0} + \mu \cdot \sigma_d \tag{1}$$

Strength of a member depends on the initial shear strength f_{v0} (cohesion), roughness of the surface μ (coefficient of friction) and vertical pressure σ_d. This model is given in HRN EN 1998-3 [1], where it is the only considered model of shear failure. Shear capacity of the wall can be determined by:

$$V_{p,Rd} = \frac{1}{\gamma_{Rd}} \cdot f_v \cdot L_c \cdot t_w \tag{2}$$

where γ_{Rd} is the partial safety factor for the resistance, L_c is the length of the masonry in compression and t_w is wall thickness. This type of failure occurs at higher vertical pressures than the loss of stability and it is characterized by the sliding of a body along a bed joint. This type of failure is characteristic of elements with low vertical pressure. Example of such an element are parts of chimneys below the roof structure, walls on higher storeys and single storey buildings.

Diagonal cracking is described by either a Turnšek-Čačovič [2] or Mann-Müller [3] failure model. The two models differ in the way diagonal crack is formed. Turnšek-Čačovič model considers that a failure occurs when the principal diagonal stress reaches tensile strength which is given with:

$$f_t = -\frac{\sigma_0}{2} + \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + (b \cdot \tau)^2} \tag{3}$$

Tensile strength of a member depends on the average vertical compressive stress σ_0, geometry factor b ($b = h/L$, but in between 1 and 1,5), wall height h and total length L and shear stress τ. Shear capacity of the wall can be determined by the following expression:

$$V_{t1,Rd} = \frac{1}{\gamma_{Rd}} \cdot L \cdot t_w \cdot \frac{f_t}{b} \cdot \sqrt{1 + \frac{\sigma_0}{f_t}} \tag{4}$$

This type of failure is usually more suited for irregular masonry, where a straight diagonal crack can form. It is also suitable for regular masonry with weak masonry units (such as adobe or unfired clay brick) and strong mortar (such as cement).

Mann-Müller model considers that a failure occurs in a stair-stepped shape, which is characteristic of regular masonry with weak mortar (such as lime mortar) and strong masonry elements (such as stone or fired clay bricks). Shear capacity in stepped failure can be determined by the expression:

$$V_{t2,Rd} = \frac{1}{\gamma_{Rd}} \cdot \frac{L \cdot t_w}{b} \cdot \left(\frac{f_{v0}}{1 + \mu_j \cdot \phi} + \frac{\mu_j}{1 + \mu_j \cdot \phi} \cdot \sigma_0 \right) \leq V_{d,lim} \tag{5}$$

where μ_j is the local coefficient of friction of the joint, if there is no more accurate data, it is given with a value of 0,6, ϕ is the clamping coefficient (the ratio of the height of the wall element and the average length of the overlap of the two wall elements). The load-bearing capacity limit at stepped failure is:

$$V_{d,lim} = \frac{1}{\gamma_{Rd}} \cdot L \cdot t_w \cdot \frac{f_{bt}}{2,3 \cdot b} \cdot \sqrt{1 + \frac{\sigma_0}{f_{bt}}} \tag{6}$$

Where f_{bt} is the tensile strength of the wall element. Diagonal types of failure are characteristic of elements with a medium level of vertical pressure. An example of such an element are walls on the lower storeys of a structure. Figure 2a) presents stair-stepped shape of diagonal failure, while figure 2b) presents straight diagonal failure of masonry wall in shear.
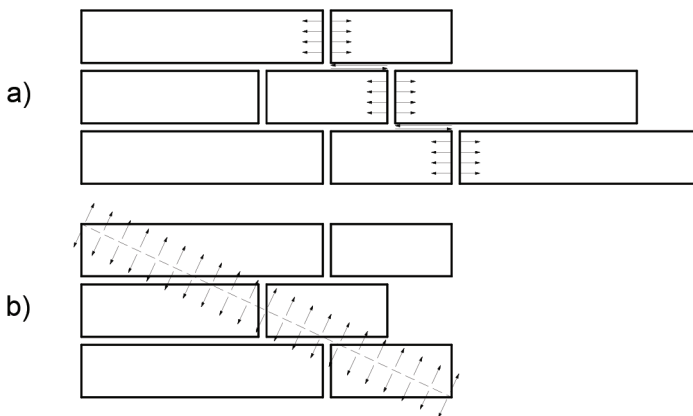
Figure 2. Diagonal tension failure modes: a) stepped b) „straight" [5]

Diagonal failure modes are more common in older masonry, so the currently valid standard HRN EN 1996-1-1 [4] does not consider this failure mode. The standard HRN EN 1998-3 [1] also does not consider this failure mode, but in the draft versions of the next generation of this standard, these failure modes are considered as mainly relevant for existing masonry.

Toe crushing may occur at high levels of vertical pressure and it is also described in [1]. Due to a combination of bending and axial pressure, the ends of a wall crush. Detailed discussion and explanation of each of the in-plane failure modes may be found in [5].

3 Strengthening of walls with non-metallic reinforcement

Although no guidelines are given in [1], non-metallic reinforcement may be used to strengthen the in-plane strength of masonry elements. FRP strips or FRCM meshes are usually used. Italian guidelines exist for design of FRP, CNR-DT 200 R1/2013 [6] and FRCM, CNR-DT 215/2018 [7] strengthening. Non-metallic reinforcement has some advantages, such as durability and better compatibility with masonry, but also disadvantages such as lower capacity and ductility and higher price. Although mentioned guidelines do provide a large amount of information and suggestions for the design of masonry strengthening solutions, some problems still exist.

Most of the problems stem from the fact that not enough research exists on the behaviour of the materials and systems used. Performance of the system relies heavily on the quality of a construction process. Unskilled labour, usage of the wrong materials or the improper technique could lead to unacceptable behaviour of the system.

For example, guidelines lack information on some of the commonly used materials throughout Europe, such as standard fired clay bricks, for which some of the parameters are not quantified (namely, safety factors for the calculation of bond length of FRP system). Regarding bond length, it is not clear how anchors or bends affect the required anchorage length.

It is also not clear from the guidelines which type of failure is avoided by a specific system. For example (figure 3), providing horizontal FRP strips with vertical spacing between them can lead to shear sliding failure between the two strips (since that part is not strengthened). It is not clear from the guidelines whether vertical strips provided at the wall ends contribute to shear sliding capacity, and to what extent. FRP material is not meant to provide dowel action if the fibres are longitudinal, so its effectiveness in that respect is questionable.
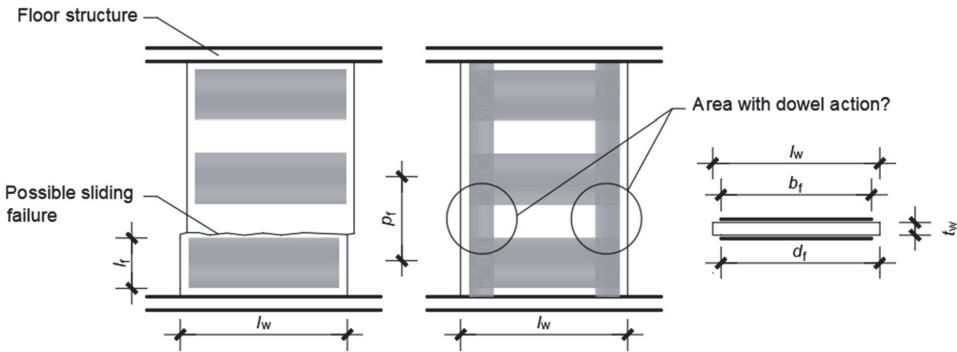

Figure 3. Sliding failure mode on the wall strengthened with horizontal strips

Regarding the spacing of the provided strips p_f it is not clear from the guidelines what is the maximum allowable spacing to ensure proper behaviour. Some guidelines should be made, to at least have one strip per possible crack position. Assuming a crack propagation at a 45° angle this would mean:

$$p_{f,max} = \min\left(0,5 \cdot h_s \; ; \; 0,5 \cdot l_w\right) \tag{7}$$

Expression (7) is not given in the norm, but it is the assumption of the authors. A similar recommendation was given in ACI 440.2R-02 [8] for the strengthening of RC elements where the maximum clear distance between the two strips is $d_f / 4$, where d_f is the distance from the most stressed strengthening fibre in tension and the most stressed masonry fibre in compression.

Expression (7) should be modified for masonry depending on morphology, masonry type and element thickness. The authors are not familiar with such an expression. Although in practical application, this condition is probably met, it is important to ensure that a possible crack is retained by at least one strip (figure 4).
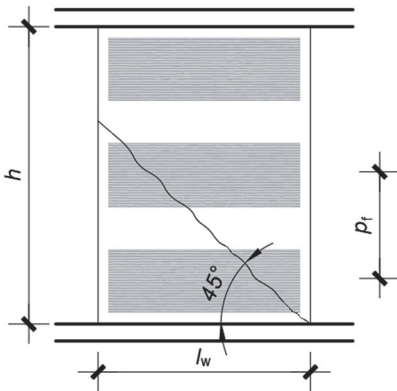
Figure 4. Sliding failure mode on the wall strengthened with horizontal strips

It is also questionable if such a failure (sliding) can occur in a realistic structure with a box-type behaviour, since the walls perpendicular to the sliding wall could slow down or possibly stop the sliding. If such a failure is possible, vertical strips or meshes should be properly anchored. Strengthening elements can either be continuous over the height of the structure and anchored into the footing or properly anchored at the storey level if they are not continuous. The anchoring is not sufficiently described and no provisions for either detailing or calculation of such elements exist. It is probably better to provide diagonal anchorage, rather than a straight vertical one since the material is strongest in tension (figure 5).
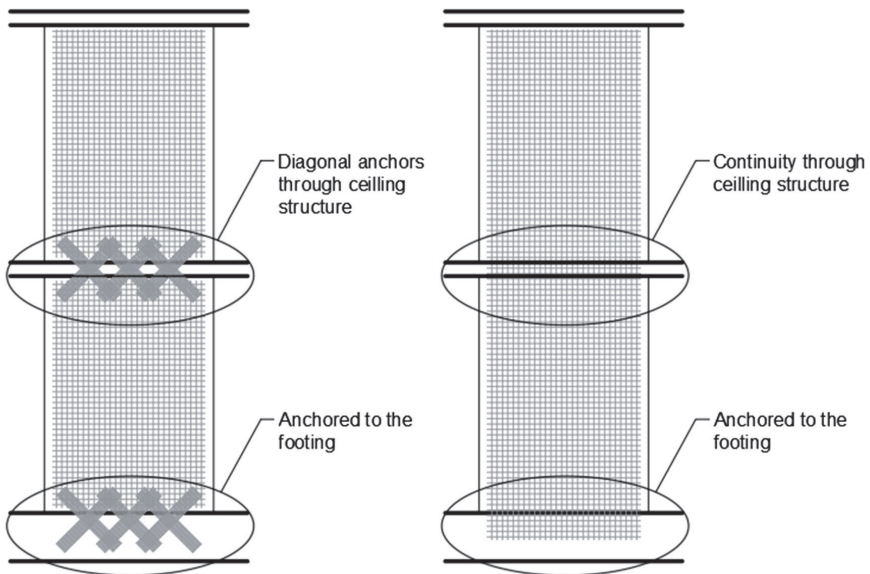

Figure 5. Anchoring of FRCM meshes through the ceiling structure

For systems to be effective for in-plane strengthening, other failure types should be avoided, and box-type behaviour ensured beforehand. The connection between the walls and between the wall and the floor should be carefully examined. The number, spacing and depth of an anchor can significantly influence the effectiveness of the connection. Although some guidelines do exist on the depth and the spacing of the anchors at the middle of the wall, it is not entirely clear as to what changes, if anything, at the connection of the two walls. Since the research is scarce, it is not yet clear what is the optimal layout of the anchors.

Since unreinforced masonry usually has a low capacity, significant strengthening is needed to ensure a satisfying level of reliability. For that reason, installing more than one layer may seem reasonable. Since more layers of the mesh can lead to the degradation of the bond, the effectiveness of multiple layers is questionable, especially if all of them are not anchored. This problem is not addressed in [7], although multiple layers of mesh seem to be allowed. Since [7] allows installing FRCM mesh without any anchors if the mesh is provided on both sides of a wall, it seems unconservative to provide more than one FRCM mesh on each side.

It is also not clear what allowable strain in the FRCM mesh should be used in a practical wall since a higher value is allowed where no debonding may occur. Since the capacity is defined for the whole wall, which usually has parts where debonding may occur and parts where it is not possible, it is not clear as to what value should be considered for a specific wall.

As the test determined the adhesion at the anchorage point, in the area far enough from the anchorage point the allowed deformation can be increased by 50 % according to [7] because the separation of the mesh can also occur at the places away from the anchorage (but at higher deformation). Of course, if the separation deformation is greater than that in which the tensile failure of the fibres $\varepsilon_{u,f}$ occurs, then $\varepsilon_{u,f}$ is taken as the allowable deformation.

Figure 5a) shows a sketch of a wall without openings and the area where minor deformation is allowed is obliquely hatched (in [7] it is not defined which is the exact area where minor deformation is allowed, but the author's assumption is that it is an area < 30 cm from all ends of the mesh because it is the bond length according to [7]). It can be seen from Figure 5a) that for this specific case a large part of the mesh cutting the crack can be activated to a greater deformation (the shaded area is small).

Figure 5b) shows a wall with an opening near the edge. For this specific case a higher allowable deformation is acceptable only for the part of the wall to the right of the opening, while for the part to the left of the opening as well as the lintel the lower value is relevant. Accordingly, it is necessary to determine the allowable deformation based on the geometry of the wall and the expected location of the crack.
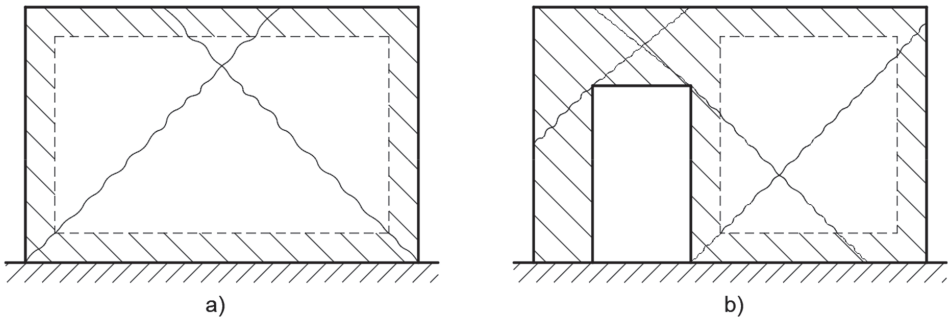
Figure 6. Display of areas with allowable deformations: a) example 1; b) example 2 [5]

4 Strengthening of walls with metallic reinforcement

Metallic reinforcement is widely used to strengthen the masonry elements. The most common methods of strengthening include shotcrete, stitching, adding concrete tie elements, or adding new walls.

These methods usually require more work and invasiveness in comparison with the non-metallic solutions. Since reinforced concrete is significantly stiffer than existing walls, a connection between the two materials might be problematic.

Shotcrete can be made on one or both sides of a wall with a thickness of around 5 cm per layer. This has a disadvantage of adding a lot of mass to the structure. An important problem is ensuring compatibility of materials because of significantly stiffer concrete parts. In most cases this means that the concrete part of the wall will provide almost all of the capacity and stiffness, while the vertical load is concentrated on the masonry part. Masonry part of the wall may crush significantly at failure. Another important thing is to ensure vertical continuity of the shotcrete wall and anchor it to the existing foundations. This might be a problem for internal walls.

The procedure of determining the thickness of the shotcrete layer is iterative because both seismic force and resistance depend on the thickness of the shotcrete layer. Since concrete is significantly stiffer than masonry, providing shotcrete only on some walls may change the global response of the structure and its eigenvectors. It may also lead to uneven requirements across the structure, meaning energy will dissipate on fewer members and high demand may arise on the connection of shotcrete walls with floors. Similar problem arises if new walls are made.

If new tie elements are provided, the question of its connection to the existing walls is questionable. Many existing walls have irregularities which make it hard to take out parts of them where concrete may be inserted. Depending on the type of floor, connection of tie columns with the floor may be more or less complex. Tie beams may be hard to cast.

Stitching is simple and not as invasive as the other mentioned methods, but it only increases the diagonal failure capacity.

5 Conclusion

The paper shows the methods of calculating the load-bearing capacity of the masonry to the in-plane shear and presents the most common methods of calculation of in-plane shear wall strengthening. The review of details that are not sufficiently well clarified or are neglected, i.e., poorly used in practice are also given. In addition to the calculation of strengthening of masonry with the reinforced concrete layer (shotcrete) and dowels, modern methods of strengthening are considered, using FRP fabrics or plates and FRCM systems.

As these calculation methods are not implemented in the existing codes, various recommendations are used based on similar assumptions, but with different input parameters and safety factors that affect the calculation result and that should be properly interpreted and used in the calculation.

Some doubts or insufficiently defined parameters that designers encounter during the calculation are highlighted. It is important to highlight such problems and to stimulate discussion in the scientific and professional community in order to improve and clarify the recommendations for the implementation of such calculations. This research is a part of the scientific project "ARES" which is described more in detail in [9]. The earthquakes that occurred in the Republic of Croatia in 2020 indicated a great need to bring the methods of strengthening masonry closer to engineers in practice.

Acknowledgements

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