REBUILDING OF A STONE MASONRY BUILDING WITH A NEW TECHNOLOGY OF FAIR-FACE DOUBLE LEAF MASONRY WALL

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Abstract: *Fair-face rubblestone masonry is a very common masonry typology in old buildings and it characterizes the architecture of many hamlets in Italy and abroad. Past earthquakes have shown its high vulnerability, especially when built with multiple-leaf masonry walls, which fail by leaf separation and/or the disintegration of individual stones under out-of-plane seismic actions. The severe damage caused by earthquakes to this type of buildings highlights the need of retrofitting them. Due to the difficulties of achieving a significant capacity enhancement, demolition and reconstruction often results the preferable option. To this purpose, appropriate technologies are needed, which are sustainable, durable, easy to realize and guarantee the seismic safety and energy efficiency typical of new buildings, safeguarding, at the same time, the architectural asset of pre-existing urban context. A new technology was developed to meet all these needs, consisting of a double leaf masonry wall: the internal leaf is made of reinforced clay masonry and the external one is made with fair-face stone masonry. The two leaves are connected through a pre-cured glass fibre reinforced polymer mesh. The effectiveness of this new technology in providing the wall with a monolithic behaviour was verified by means of shake table tests (topic of a dedicated paper in the present Conference). This paper presents the field application of the proposed innovative technology to a two-storey building located in a high seismic hazard site, which underwent reconstruction and seismic/energetic upgrade. The design phases are described, considering global and local analyses carried out to check that monolithic behaviour is ensured under out-of-plane seismic actions. The case study shows the effectiveness and the high safety level of the new technology developed for fair-face rubblestone masonry in earthquake prone areas.*

1. Introduction

Fair-face rubblestone masonry is a very common masonry typology in old buildings and it characterizes the architecture of many hamlets in Italy and abroad. Past earthquakes have shown its high vulnerability, especially when built with multiple-leaf masonry walls, which fail by leaf separation and/or the disintegration of individual stones under out-of-plane seismic actions (Figure 1). This is the case of many buildings hit by the recent Central Italy seismic sequence (de Felice et al. 2022, Borri et al. 2019), but the same collapse modes have been envisaged after many other destructive earthquakes as reported in Göçer (2014), Sherafati and Sohrabi (2016), Brando et al. (2017), Marotta et al. (2017), Penna (2015).

The 2022 Turkey-Syria earthquake also had a profound impact on rubblestone masonry buildings in the affected regions. The powerful earthquake ground motions resulted in widespread cracking, displacement, and, in some cases, complete structural failure of these buildings, posing a substantial risk to both human safety and cultural heritage conservation. This unfortunate outcome underscores the ongoing challenge of safeguarding such vulnerable architectural treasures against seismic events and emphasizes the urgency of implementing retrofitting strategies and improved construction practices (Smith et al. 2022; Jones and Brown 2022).

The severe damage caused by earthquakes to this type of buildings highlights their vulnerability to seismic actions. Achieving a significant increase of their seismic capacity through interventions is, however, difficult. Thus, the preferred option is often demolition and reconstruction. To this purpose, appropriate technologies are needed, which are sustainable, durable, easy to realize and guarantee the seismic safety and energy efficiency typical of new buildings, safeguarding, at the same time, the architectural asset of pre-existing urban context

A new technology was developed to meet all these needs, consisting of a two-leaves masonry wall: the internal leaf is made of reinforced clay masonry and the external one is made with fair-face stone masonry. The two leaves are connected through a pre-cured glass fibre reinforced polymer (GFRP) mesh. The effectiveness of this new technology in providing the wall with a monolithic behaviour was verified by means of shake table tests in the framework of a research project lead by Roma Tre University, together with ENEA and University of Roma Sapienza, with the support of Regione Lazio and the participation of Fibre Net and Consorzio POROTON® Italia, as described in a companion paper (Sangirardi et al., 2024).

Figure 1. Damage scenarios after the Central Italy earthquake sequence (2016-2017) in the municipality of Accumoli (Rieti).

2. The building and the materials

This paper presents the field application of the proposed innovative technology to an old building located in the municipality of Vallesaccarda in the province of Avellino (Italy), which is a well-known high seismic hazard area. The existing stone masonry building results in state of neglect and it is characterized by the typical structural deficiencies of this type of building, such as the poor quality of the masonry arrangement and mortar, disjointed and deteriorated timber floors, lack of adequate construction details.

In order to guarantee the seismic safety and energy efficiency typical of new buildings, and considering that the aggregate to which the unit belongs is in an abandoned state, demolition and reconstruction was considered the preferable option adopting the technology developed in the research recalled in the introduction.

The reconstruction project involved the realization of a tourist accommodation building on two storey with eaves and ridge heights of 5.8 m and 7.3 m respectively, with a footprint of 10.4 m x 8.4 m in addition to the external staircase for access to the upper floor (Figure 2 and Figure 3).

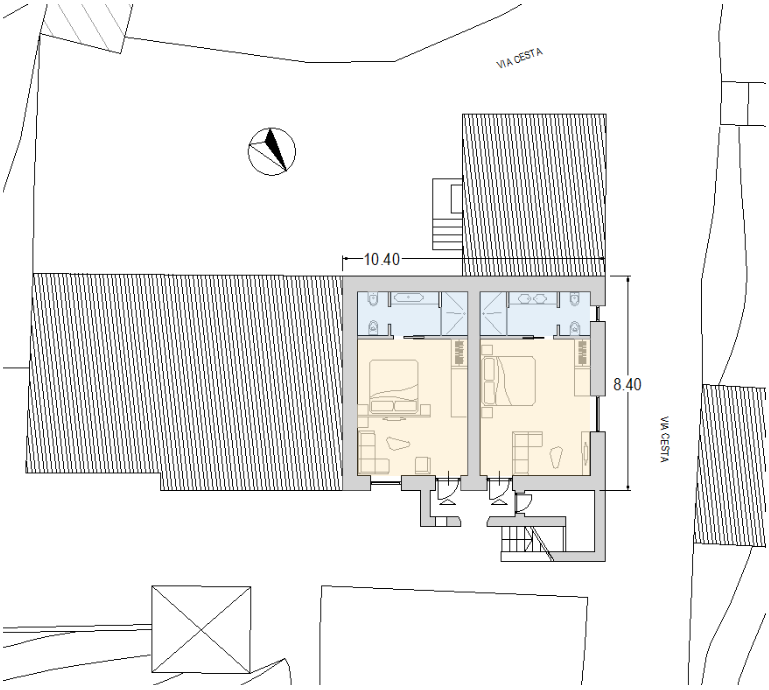

Figure 2. Building layout.

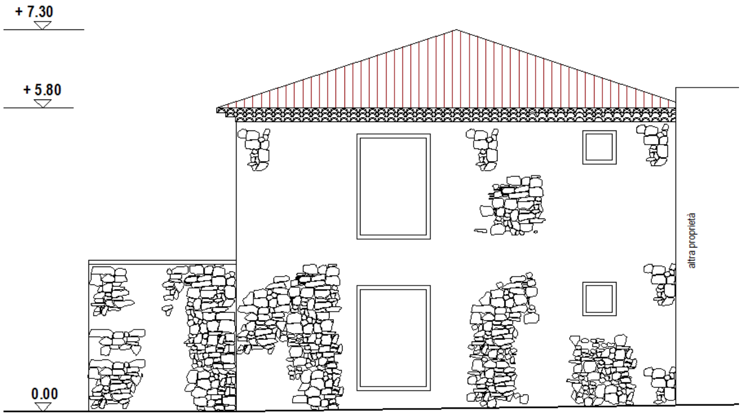

Figure 3. North-west elevation.

The structural system is based on load-bearing reinforced clay masonry walls, resting on a reinforced concrete foundation slab with emerging beams at the bottom of the walls. The reinforced clay masonry walls are connected through reinforced concrete beams to the first slab and to the roof. The first slab is made with a timber-concrete composite floor (12 cm x 20 cm glued laminated timber beams and 5 cm of reinforced concrete top). The roof is made with glued laminated timber beams (18 cm x 28 cm for the ridge and 12 cm x 20 cm for the other beams) and double cross plank.

The fair-face double leaf masonry adopted for the external walls of the project is constituted by (Figure 4):

- the load bearing back face, made of 40 cm thick reinforced clay masonry, providing adequate mechanical and seismic resistance, thermal-acoustic insulation and entailing simple and rapid construction. This reinforced brickwork leaf has an average unit weight of 11 kN/m³.
- the external wall, made of 25 cm thick fair-face stone masonry and M10 natural hydraulic lime-based mortar. The stones to be laid can be collected after demolition of existing buildings to mitigate construction cost, associated to both rubble disposal and supply of new materials. This fair-face stone leaf has an average unit weight of 25 kN/m³;
- Connection between wall leaves, by means of the same lime-based mortar used for the front façade and placing, in alternated mortar joints (i.e. those in which horizontal steel stirrups are not present), a specifically designed FGRP mesh with a large mesh size aiming at inducing a monolithic behaviour of the masonry and preventing the stone facing from crumbling due to seismic actions.

a)  b) 

Figure 4. The new technology of fair-face double leaf masonry: a) section view, b) detail of the front face and stone laying.

The reinforced masonry walls are made by using insulation filled clay blocks produced by Fornaci Laterizi Danesi (thickness equal to 40 cm and percentage of voids equal to 45% filled with insulation, Figure 5). This type of blocks allows to achieve the thermal insulation required for the project for the external walls without placing any additional thermal insulation layer, finishing the internal face of the wall with common plaster. The geometry of the block allows the construction of corners without additional block types. Moreover the blocks can be easily laid after the vertical steel reinforcement is positioned. A cementitious mortar class M10 (compression strength not lower than 10 N/mm²) is used to lay the units and also to fill the vertical cavities where the vertical reinforcement is located. The reinforcement is constituted by high ductility ribbed steel rebars of B450C type. The diameters of vertical rebars and horizontal stirrups are 16 mm and 8 mm respectively. The distribution of the reinforcement was defined by design and according to the minimum quantity required by national code ($0.04\% \leq \rho_h \leq 0.5\%$ and $0.05\% \leq \rho_v \leq 1.0\%$ where ρ_h and ρ_v are the horizontal and vertical reinforcement ratio measured with respect to the gross cross-sectional area of the wall). Consequently, the horizontal reinforcement is constituted by 2ϕ8 placed in alternating mortar joints, thus with a 40 cm spacing, resultin in a reinforcement ratio of about 0.06%. The vertical reinforcement is constituted by 1ϕ16 placed at the end of each wall, at the intersection between walls and spaced about 100 cm (Figure 6), the corresponding reinforcement ratio is between 0.05% and 0.15%.

The reinforced clay masonry walls are characterized by the following mechanical properties: characteristic compressive strength, f_k = 4.92 N/mm², characteristic initial shear strength, f_{vk0} = 0.30 N/mm², modulus of elasticity, $E = 1000 \cdot f_k$ = 4920 N/mm², shear modulus $G = 0.4 \cdot E$ = 1968 N/mm², partial safety factor for seismic design, γ_m = 2,4 (NTC 2018).

The GFRP mesh is produced by Fibre Net with a large mesh size (132×132 mm and nominal diameter of ~ 4 mm, it varies a little in warp and weft directions) It is designed to be sufficiently deformable to be accommodated around the stones of the external facing wall. During the construction of the reinforced clay masonry wall the mesh is placed into the mortar bed joints, alternating the mesh and the horizontal rebars of the reinforced masonry (Figure 7), and part of the mesh sheet is left protruding. The protruded portion of the mesh is subsequently incorporated into the external stone masonry during the its construction (Figure 4b).

For the single bundle of the mesh (warp direction) the following mechanical properties were obtained in mechanical characterization laboratory tests: tensile strength, F_{fu} = 5.6 kN, modulus of elasticity, E_f = 15.5 GPa and ultimate strain, ε_{fu} = 3.3 %. Pull-out tests executed on single strands allowed to verify that an embedded length of 16 cm is enough to prevent the slip between fibres and matrix both in cement and lime mortar (Sangirardi et al., 2024).

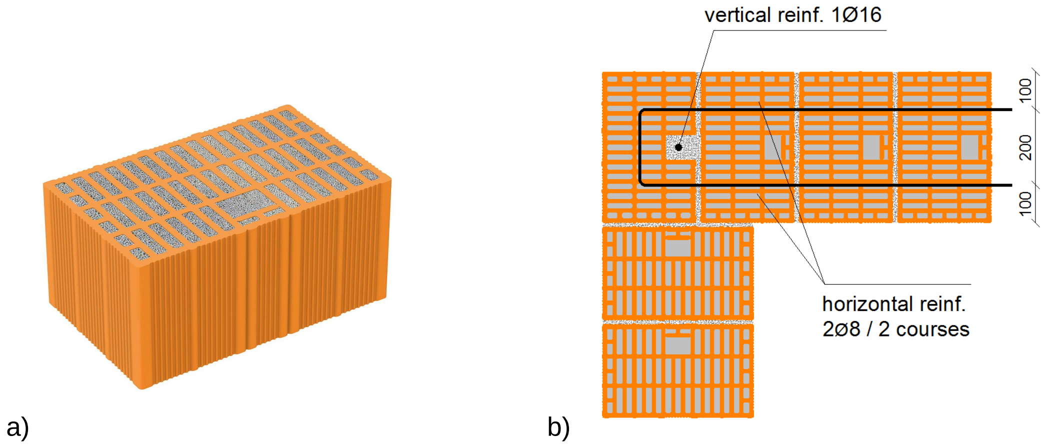

Figure 5. Reinforced masonry made with insulation filled clay blocks: a) solid view of Normablok® Più S40 MA and b) wall corner detail.

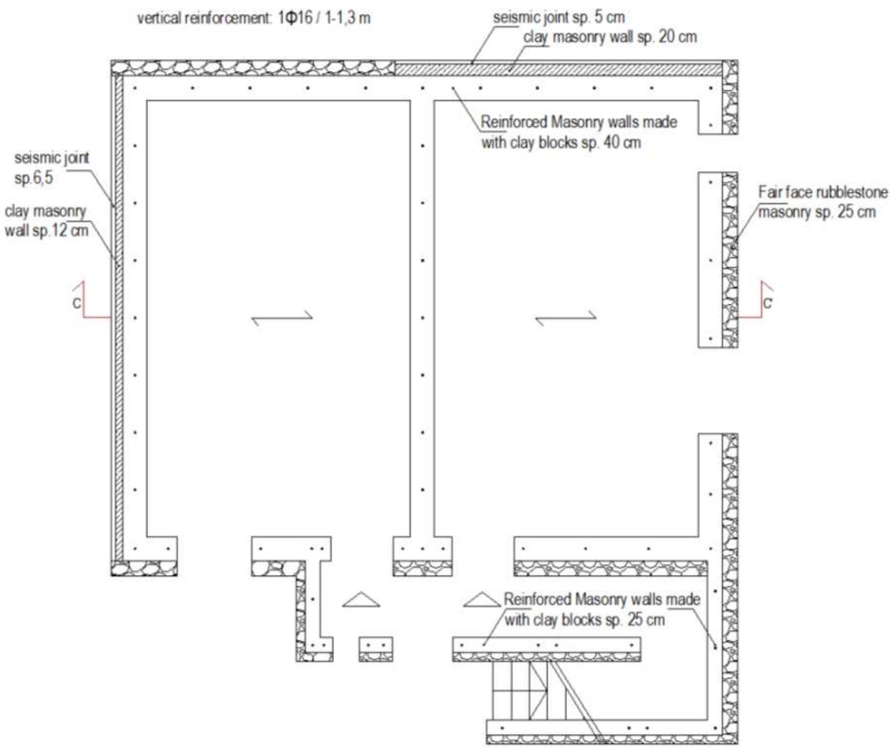

Figure 6. Ground floor structural plan.

Figure 6 shows the ground floor structural plan: load-bearing reinforced masonry walls and the external fair-face stone masonry are visible. At the two adjacent buildings, the realization of seismic joint of 6.5 cm towards the south-east building and 5 cm towards the south-west building was envisaged, giving full structural independence to the new unit under reconstruction. The reinforced masonry walls plan density, net of staircase, is higher than 7.5% along each principal direction, which is quite high density typical of old buildings characterized by thick walls and short slab span.

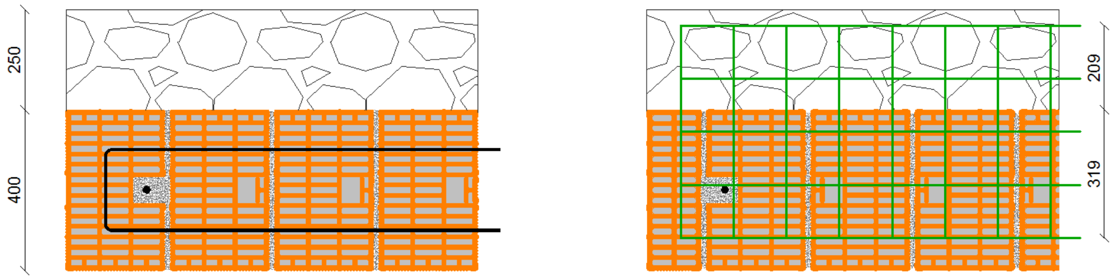

Figure 7. Alternating reinforcing bars and fiberglass mesh in the mortar bed joint of the reinforced masonry.

3. Seismic design

3.1. Design of the structure

The building, according to the intended use, was designed for a nominal service life V_N = 50 years, which corresponds to a probability of exceedance of 10% for life safety limit state with a consequent return period T_R = 475 years. The building is in a substantially flat area and a medium soft soil, which gives a topography category T_1 and soil type C, according to the national code (NTC 2018). The elastic response spectrum and the design spectrum at life safety limit state are shown in Figure 8. The parameters defining the seismic action have been calculated with the software "Spectra-NTC" provided by the Consiglio Superiore dei Lavori Pubblici (Board of Public Works).

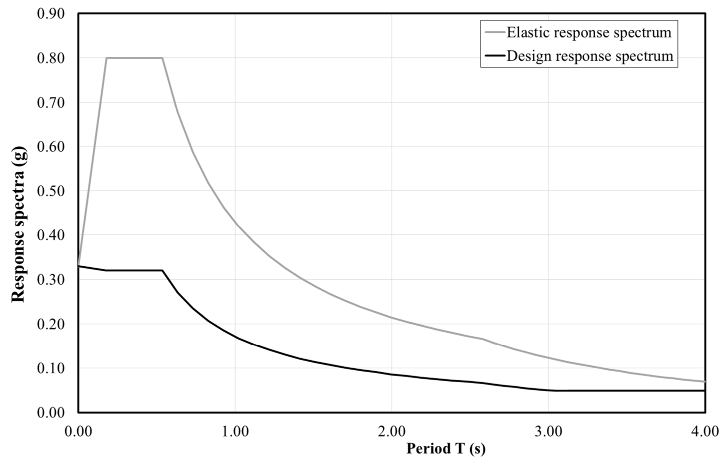

Figure 8. Elastic response spectrum (a_g=0.245g, F_0=2.429, T_C=0.368s, C_C=1.461, S=1.343) and design response spectrum at life safety limit state.*

The seismic design of the building was performed adopting a modal response spectrum analysis, which allows to design the reinforced clay masonry walls, already shown in Figure 6, considering the global behaviour of the building. Subsequently, local mechanisms were analysed only to check the two-leaves bond under out-of-plane seismic actions.

The behaviour factor, according to national code (NTC 2018) is $q = q_0 * K_R$. For reinforced masonry buildings is $q_0 = 2.5*(\alpha_u/\alpha_1)$. Considering the presence of the external staircase, the building is classified as nonregular in plan and in elevation, therefore (α_u/α_1) is to be assumed equal to 1.25 and the K_R factor is equal to 0.8. Consequently, the applied behaviour factor for the building under study is q = 2.5.

The modal response spectrum analyses were carried out by means of a FE model in which the masonry walls were modelled as shell elements, the first slab was modelled as rigid shell, whilst the reinforced concrete beams on the top of the walls and the timber beams of the roof were modelled as beam elements. The roof was finished with a double plank modelled as shell elements rigidly connected to the beams. The beams of the roof were hinged, assuming the transmission of bending through connections to be negligible. Masonry were fixed at the bottom and rigidly connected to the reinforced concrete top beam. A stiffness reduction of 50% was assumed for all elements of the model and distributed mass was assumed, in order to account for the effect of the additional stone masonry mass.

Figure 9 shows a view of the model used for the analyses together with the first vibration mode in X direction (parallel to the ridge) at 0.084 s involving 63% of mass, whereas the first vibration mode in Y direction (perpendicular to the ridge) is at 0.066 s involving 64% of mass.

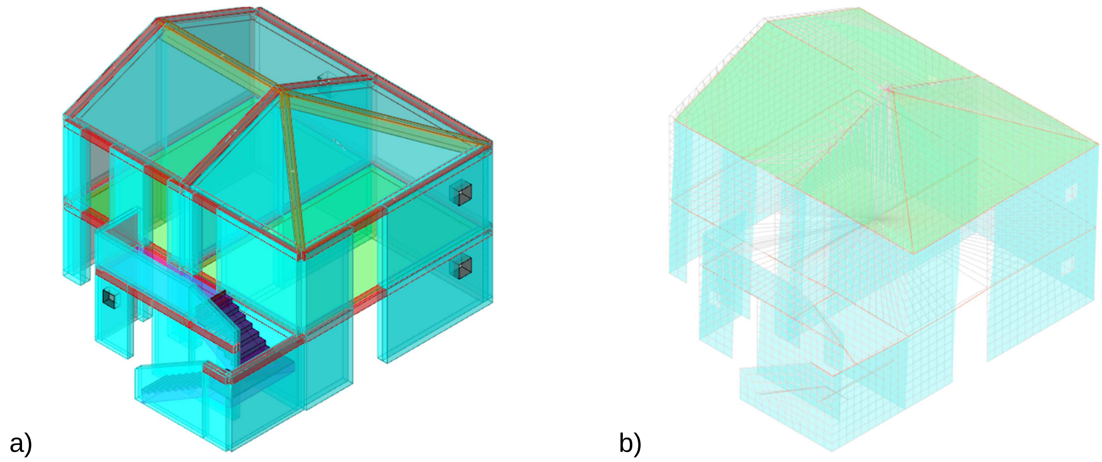

Figure 9. a) FE model and b) first vibration mode.

The effects of the seismic action were obtained through complete quadratic combination (CQC) of the effects related to the first 20 vibration modes, which allow reaching a total participation mass of 98%.

The safety coefficient obtained for the walls, by means of the modal response spectrum analyses, moves from a minimum of 2.4 to a maximum of 9.1. The analyses show that the large additional mass of the fair-face rubblestone masonry results adequately supported by the reinforced clay masonry walls, in terms of global behaviour of the building. Also in terms of local behaviour, accounting for the out-of-plane seismic actions, the reinforced clay masonry walls is capable to support the additional mass of the stone masonry with safety coefficients included in a rage from a minimum of 5.8 to a maximum of 20.9.

3.2. Design of the fair-face stone masonry connection

The connection of the fair-face rubblestone masonry to the back reinforced clay masonry was verified by means of two different local analyses accounting only for the tensile strength developed by the fiberglass mesh.

The first verification carried out consisted in evaluating the inertia force induced by a portion of stone masonry whose dimensions were set equal to the influence area correlated to each transversal fiberglass bundle of the mesh embedded in the mortar joints (Figure 7). The volume correlated to each bundle is equal to 13.2 cm (mesh spacing) x 40 cm (height of two course of clay units) x 25 cm (thickness of the fair-face stone masonry) equals to 0.0132 m^3. From the density of the stone masonry equals to 25 kN/m^3 and considering the maximum accelerations of the elastic spectra (a_g*S*F_0) at life safety limit state the following force results: F_{SLV} = 264 N. It is easy to observe that the maximum out-of-plane seismic force induced by the portion of stone masonry to a single bundle of fiberglass is one order of magnitude lower than its tensile strength (F_{fu} = 5.6 kN), therefore the verification is fully satisfied.

The second verification consists in applying the analysis of the possible collapse mechanisms. The activation multiplier of three out-of-plane mechanisms (Figure 10), considered particularly significant for the case under study, were compared, namely:

1. Overturning mechanism of a wall (the one relating to the second floor) with a height of 2.9 m.
2. The out-of-plane bending mechanism involving the entire height of the wall equals to 5.8 m.
3. Overturning mechanism of the entire wall with a height of 5.8 m.

The analytical evaluations of the collapse mechanisms were made setting a rate equal to 10% of the maximum force experimentally attributed to GFRP connectors, therefore equal to 0.56 kN, for the benefit of safety. Along the 2.9 m height, 7 GFRP meshes are distributed in alternated mortar joints, as already said with a spacing of 40 cm. Assuming a width of 1 meter, 7.5 transversal fiberglass connectors are distributed along the wall since the mesh size is 132x132 mm. Therefore, for each joint reinforced with mesh, a force equal to $F_{tot,mesh}$= 4.2 kN was assumed.

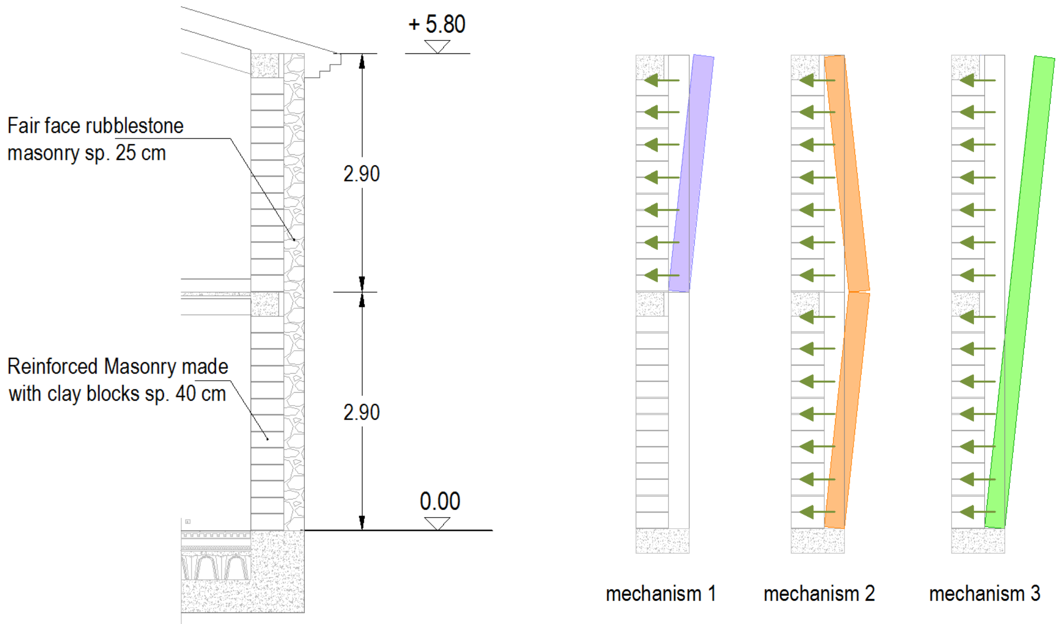

Figure 10. Vertical section of the external double-leaf masonry wall and schemes of the significant collapse mechanisms.

In the case of mechanisms 1 and 3, the activation multiplier α has been calculated by solving the following equation, which expresses the equilibrium between the overturning and the stabilizing moment:

$$\alpha_0 P \frac{H}{2} = \sum_i F h_i + P \frac{t}{2} \tag{1}$$

where P is the weight of the facing of height H equal to 2.9 m or 5.8 m, t its thickness, F is the value of the horizontal force attributed to each joint reinforced with mesh distributed at the heights h_i.

For the mechanism involving only the 2.9 m high wall the value of the multiplier is equal to 1.65 g (mechanism 1), whereas for the mechanism involving two decks, with a total height equal to 5.8 m the multiplier is equal to 1.63 g (mechanism 3). Therefore the verification are fully satisfied for both the damage limit state and the life safety limit state, adopting a linear approach with behaviour factor q = 2. The acceleration for damage limit state is 0.117 g and for life safety limit state is 0.165 g to be assumed as reference for the mechanism 3, where the wall involved lays at the ground. The spectral acceleration at the fundamental period of the structure for damage limit state is 0.178 g and for life safety limit state is 0.248 g to be assumed as reference for the mechanism 1, where the wall involved is not laying at the ground.

It is noteworthy that the multipliers for the same cases in the hypothesis of the absence of the fiberglass mesh would have been equal to 0.086 g and 0.043 g respectively. It follows that the verification for both the damage limit state and the life safety limit state would not have been satisfied.

For the mechanism 2 (vertical bending on the entire height of the wall), the multiplier has been calculated applying the equilibrium according to the principle of virtual works. The multiplier is equal to 0.172 g in absence of the transversal connectors and is equal to 1.79 g accounting for the contribution of the transversal connectors as reported in Figure 10. The capacity developed by mechanism 2 also without the connectors is enough to overcome the required acceleration at life safety limit state, already saw for mechanism 3. However it is evident the huge positive contribution of the connectors which means a significant increase of the safety level.

4. Conclusions

Demolition and reconstruction often results the preferable option when intervening on fair face rubblestone masonry buildings with the purpose to obtain a consistent seismic/energetic upgrade in order to satisfy the requirements of new buildings. This because rubblestone masonry buildings are often very heavily damaged during earthquakes and a rehabilitation is practically impossible or, sometimes, retrofitting strategies might be more time consuming and would possibly not upgrade the performance of the buildings up to the required standards, especially in highly seismic prone areas.

In the framework of a research project lead by Roma Tre University, a new technology, to be used for reconstruction, was developed with the purpose to be sustainable, durable, easy to realize while ensuring the seismic safety and energy efficiency typical of new buildings, safeguarding the architectural asset of pre-existing urban contexts. The developed technology consists of a two-leaves masonry wall: the internal leaf made of reinforced clay masonry and the external one made with fair-face stone masonry, connected through a glass fibre reinforced polymer mesh.

The proposed double leaf masonry wall technology was applied to a two-storey building located in a high seismic hazard area, which underwent reconstruction and seismic/energetic upgrade. The high safety level obtained for the structure from global and local analyses shows that the reinforced clay masonry walls are adequate to support the additional mass of the stone masonry. The connection of the fair-face rubblestone masonry to the back reinforced clay masonry has been verified accounting only for the strength of the fiberglass mesh and using two local analyses: evaluating the inertia force of the stone masonry portion correlated to one transversal fiberglass bundle and applying three typical collapse mechanisms. For all the cases considered in the analyses, the obtained high safety level shows that the GFRP mesh used to connect the two leaves of the wall effectively provides it with a monolithic behaviour.

5. References

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