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# Push ‘o ver: a pushover test program on an existing brickwork construction

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**Abstract**

An experimental pushover was carried on two very similar unreinforced masonry constructions. These constructions were obtained from an existing two-story brickwork building, located at Castel di Lama and to be demolished due to the damage suffered during the seismic sequence that affected central Italy from August 2016 to January 2017. The central stairwell was demolished in a controlled manner and, thanks to the symmetry of the original building, two almost identical portions were delivered: one was simply repaired, by repointing the cracks caused by the earthquakes, the other was seismically strengthened using Fibre Net reinforced plaster or composite reinforced mortar. The plaster was applied on both faces of the load-bearing walls of the first level and only on the external face at the second level. Preliminary dynamic characterization tests were carried out on both constructions. Then, pushover tests were carried out until collapse on both portions, using a steel structure placed on a reinforced concrete mat supported by four piles, which allowed to apply increasing horizontal forces at the two floors. The experimental results were analysed carefully and compared with those obtained from suitable numerical models.


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*Keywords:* masonry existing buildings; full-scale pushover test; experimental in situ test; pushing test.

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## 1. Introduction

“Push 'o ver" is an English-Neapolitan expression that literally means “push in real”. It was chosen to summarize a project in which an experimental pushover was carried on two very similar masonry constructions. The objective of the project was the experimental evaluation of the improvement in terms of seismic capacity, obtained by means of an innovative structural reinforcement technique. Two very similar real buildings, one not retrofitted and the other retrofitted, were subject to push tests up to collapse, carrying out a push-over on the field, normally performed numerically with suitable codes.

It is well known that the pushover analysis is one of the most used numerical techniques for the evaluation of the seismic capacity of a structure. It cannot substitute a step-by-step dynamic analysis but allows pointing out the main features of the behaviour under horizontal forces of a structure and its weak points. Relevant analogues previous studies are in the bibliography (Aldemir et al. 2018, Cobanoglu et al. 2017, Candela et al. 2015, Chourasia et al. 2016, Hogan et al. 2018, Hogan et al. 2015, Moon et al. 2007).

The two buildings were obtained from an existing two-story brickwork building, located at Castel di Lama, Italy, to be demolished after the damage suffered during the seismic sequence that hit central Italy from August 2016 to January 2017. The central stairwell was demolished in a controlled manner and, thanks to the symmetry of the original building, two almost identical portions were delivered: one was simply repaired, by repointing the cracks caused by the earthquakes, the other was seismically improved using Fibre Net reinforced plaster or composite reinforced mortar. The plaster was applied on both faces of the load-bearing walls of the first level and only on the external face at the second level. The first solution certainly guarantees a better result; the second one allows a degree of seismic upgrading without the inhabitants having to abandon their dwellings and is suitable when masonry is not prone to disintegration. In both buildings the first floor was strengthened, while a steel reticular structure replaced the original roof; the masses of the roof were simulated by placing water tanks at the top. Dynamic characterization tests were first carried out on the two buildings by surveying environmental. Then forced vibrations tests were performed by dropping a mass near the buildings, from different heights. Finally, thrust tests were performed on the two bodies separately, using a special steel contrast structure and two hydraulic jacks. The foundations for the contrast structure were made with a plinth in reinforced concrete on 4 piles (diameter 800 mm).

In this paper the various phases of the project are described, while the experimental and numerical results are the subject of parallel contributions (Addessi et al. 2022, Zona et al. 2022).

## 1 The buildings

The original structure, built in several phases in the first half of the nineteenth century, had a rectangular plan of 11.42 x 10.87 *m* (Fig. 1). A reinforced concrete small room, recently built, leaning against the main building with levels offset from it, was demolished before the start of this study. Therefore, in plan, two almost symmetrical portions joined by the central staircase were distinguishable.

The building was composed of two floors above ground, an attic and an underground floor that occupied only a limited portion of the structure. The supporting structure was in masonry with brick elements and mortar. The original staircase had been replaced with a new one in reinforced concrete, on the ground and first floor, while the original one with a wooden structure remained at the basement. The first deck was made of prefabricated concrete beams and bricks between them. The beams are spaced of 60 *cm* and have a width of 12 *cm* and a height H=16+4 *cm* (brick + concrete slab). The deck in the rooms of the second storey were made of IPE 140 beams, spaced of 80 *cm*, and bricks; in the corridor, in correspondence of the stairwell, had been substituted in the 1960s. The third deck was present only in the central rooms, while in the lateral areas there was a false ceiling in *camorcanna*. The attic in correspondence with the corridor, renovated in the 1960s, was vaulted with reinforced concrete casting. The roofing was in wood and pushing on the walls. The foundations were in masonry.

Due to the earthquake that affected central Italy since August 2016, the original building suffered damage in various parts. It was classified as unusable and, therefore, to be demolished and rebuilt. Taking advantage of the almost symmetry of the building, it was decided to demolish the central staircase body in a controlled manner, thus obtaining two portions that were almost equal from a structural point of view (Fig. 2). These will be referred as to building 1 and building 2 in the following.

Fig. 1. The original building: view, plan and vertical section.

On the two separate portions of the building, interventions were carried out to make the floors rigid. At the first floor an extrados reinforced concrete slabs was realized, while a truss structure was inserted at the second one. These interventions were necessary also to ensure an appropriate distribution of the horizontal forces, applied at one end of each building, to the vertical masonry walls.

Building 1 was just repaired, while building 2 was seismically improved using the Fiber Net system. In both cases, the roof load, estimated at approximately 1.5 $kN/m^2$, was simulated using plastic tanks filled with water.

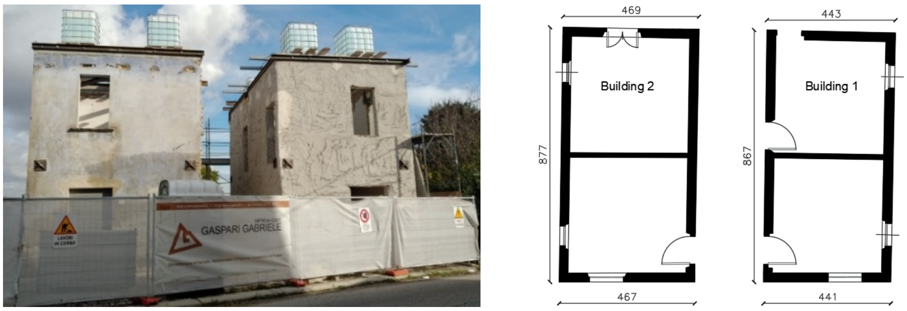


Fig. 2. Building 1 (not consolidated, on the left) and building 2. Plans of the first floors.

## 2 The knowledge of the structure

The diagnostic investigation was divided into the following phases.

- Survey of the structural geometry;
- Identification of construction details for masonry structures (quality of connections between vertical walls and between floors and walls; presence of curbs and architraves, presence of highly vulnerable elements, type of masonry and its construction characteristics); the checks are based on visual surveys, carried out generally by means of tests of the types of floor and masonry and between orthogonal walls and floors in the walls;
- Determination of the mechanical properties of masonry. Tests with double flat jacks and mortar characterization tests (type of binder, type of aggregate, and physical / mechanical characteristics of stones and / or bricks). A test is recommended for each type of masonry present;
- Determination of the mechanical properties of the materials of the reinforced concrete structures, not of interest to the project.

The demolished portion of the building was used for the following in situ tests on walls: i) Tensile tests for bending and compression tests on several brick specimens; ii) Single and double flat jack tests to determination of the stress state and of the deformability and strength characteristics of the masonry; iii) Diagonal compression tests on non-reinforced masonry (Fig. 3).

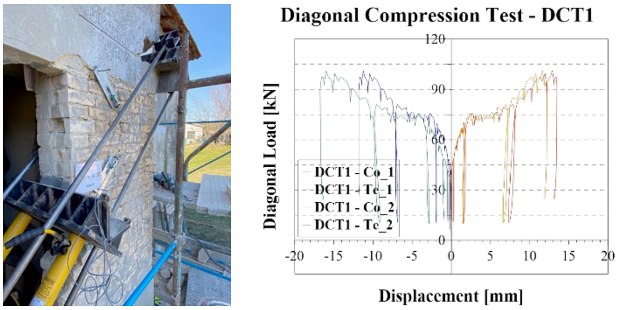


Fig. 3. Diagonal compression test setup and load – diagonal displacement diagrams.

The main results are summarized as follows: i) The compression tension in masonry was about 0.29 $N/mm^2$ at the measure point; ii) The elastic modulus of masonry was 1278 $N/mm^2$; iii) The tension at the first crack was 2.0 $N/mm^2$; iv) Diagonal: $f_{t,RILEM}$= 0.228 $N/mm^2$; $f_{v0}$= 0.152 $N/mm^2$; G=748 $N/mm^2$; E= 2135 $N/mm^2$. These values were considered for the sizing of the test apparatus and to calibrate the Fiber Net CRM consolidation system in order not to cause excessive increases in strength and stiffness.

## 2. Fibre Net CRM system application

The application of the Fibre Net system consists in the following steps (Fig. 4).

1. Evaluation of the starting phases. Study of the masonry in its initial conditions, in terms of geometry (type of blocks, thickness, presence of different layers, types of joints, etc.) and materials (origin of the blocks, joints of mortars, eventual plaster mortar, eventual presence of diatons, etc.).
2. Removal of eventual existing plasters. Removal of existing plaster and defective parts. Once removed the plaster, it is necessary to wash the outer layers of masonry using a high-pressure cleaner that can obtain a scarification of the joints and to remove the surface portions of disaggregated and non-cohesive mortar so that it can be brought back to the face plane as far as possible. Such action will allow the mortar to penetrate the masonry joints and improve adhesion. Remove the powder coat still present on the wall surface. Washing must be done from top to bottom of the façade. If required, re-build the masonry where badly damaged or missing.
3. Wetting the to be plastered surface. Prior to laying the mortar it is necessary to wet the masonry, the substrate will have to be saturated but without surface water stagnation.
4. Initial stretch coat laying. Under certain conditions, it may be necessary to apply a scratched coat layer. These needs are dictated by the conditions of the wall support and the performance characteristics (in fresh condition and even hardened) of the type of mortar to be used. In this case, apply a scratched coat of mortar to full cover the support, making a thickness of 5–10 mm. Push the rib upwardly using a toothed spatula (measuring 10 mm). This creates a rough and irregular surface that ensures perfect cohesion of the next layer of mortar coating. It is recommended to lay the rib with the right consistency to obtain a perfect roughness.
5. Prepare the holes for slab-connections. The connection of reinforced mortar system to concrete slabs at base or top of the wall is needed. Circular section improved adhesiveness rebars in AISI 304 are used, orientated in vertical direction on the surface of the masonry wall to be reinforced. Bar diameters, as well as inter-bar spacing, are determined based on the structural analysis (ø8 mm). The bar spacing distance equal to 0.40 m between bars is recommended. Perform the holes with vertical direction using a rotopercutor driller. The hole diameter should be more or equal to 1.5 times the nominal diameter of rebar. In the case of connection to RC structures of the Ground Floor the hole depth should bring an anchorage length of 50 times the nominal diameter of rebar, while in the case of connections between walls situated on contiguous levels, the hole will pass through the slab. Clean the inside of the holes by removing the resulting powder from the perforation by means of a pressurized air jet.
6. Installation of slab-connections rebars. Perform the injection of resin into the holes. This injection shall ensure to reach the deepest portion of wholes. Insert the bar into the hole up to the maximum depth, applying a bar rotation around its own axis, in such a way to obtain an optimal distribution of injected resin. The complete filling of the

cavity is ensured if some leakage of resin is observed on surface in the final phase of rebar introduction. The free bar anchorage depth to be lain in mortar thickness will be at least 50 times the nominal diameter of rebar, or 30 times if the bar is posed with a “L” shape. Rebars are to be placed in such a way to remain in contact with the wall surface to be reinforced. Before to proceed with mortar application is necessary to wait for complete injection resin hardening.

7. Prepare the holes for the connectors. Wait for at least 24 hours to ripen the scratched coat before proceeding with subsequent operations. Mark the position of the connectors on the face of the wall depending on the size of the reinforcement (the number of connectors per square meter and the average distance between them). Execute the holes for the cross connectors with the rotate-percussion type drill. Caution: the use of percussion on very badly built masonry can cause significant disruption; in this case proceed with the drilling by simple rotation. The holes, from the diameter of approx. 12 mm, must cross the entire thickness of the masonry so that the two connectors are inserted, which will be inserted on one side of each side: a "long" connector and a short connector, the length of which is chosen in such a way as to ensure a minimum overlap of 10 cm. Expand the hole diameter on that face of the wall where "short" connectors will be inserted, for a length slightly higher than that of the "short" connector. This extended widen portion will allow the correct overlapping of the two connectors in that area. Clean the inside of the holes by removing the resulting powder from the perforation by means of a pressurized air jet.
8. Placement of the mesh and insertion of the connectors. Place the mesh on the face intended for inserting the "long" connectors (the face in which the holes have not been enlarged). Place the stress distributor device on one of the holes provided for the "long" connectors and insert the connector by holding stable the device. Repeat the operation for the different holes / connectors. Place the mesh on the other side of the wall (face intended for inserting "short" connectors, whose holes have been enlarged). Make resin injection in the enlarged holes of the holes. The injection must guarantee full filling of the extended portion and at least partial penetration in the not-enlarged area. Place a centred stress distribution device on the hole and insert the connector by holding the first stable and penetrating the resin previously injected.
9. Application of the mortar coating. It is necessary to wait for the complete hardening of the connective injection resin before proceeding with the application of the mortar coating. When dust or debris is produced during mesh placement, proceed to the perfect cleaning of the scratched coat before laying the defined mortar coating. Depending on the type of mortar used and the overall thickness to be achieved, the plaster can be applied in one layer (normal condition for thicknesses up to 35 mm) or in two or more subsequent layers. During application, ensure the correct positioning of the mesh in the middle of the thickness. Ensure a damp maturation of mortar avoiding intense sunlight or ventilation and wetting at least twice a day for 7 days, beginning 24 to 48 hours after laying. Wait at least 10 days before laying any eventual finishing layers. Paintings or coloured coverings may only be applied after the mortar maturing and no later than 28 days after laying.

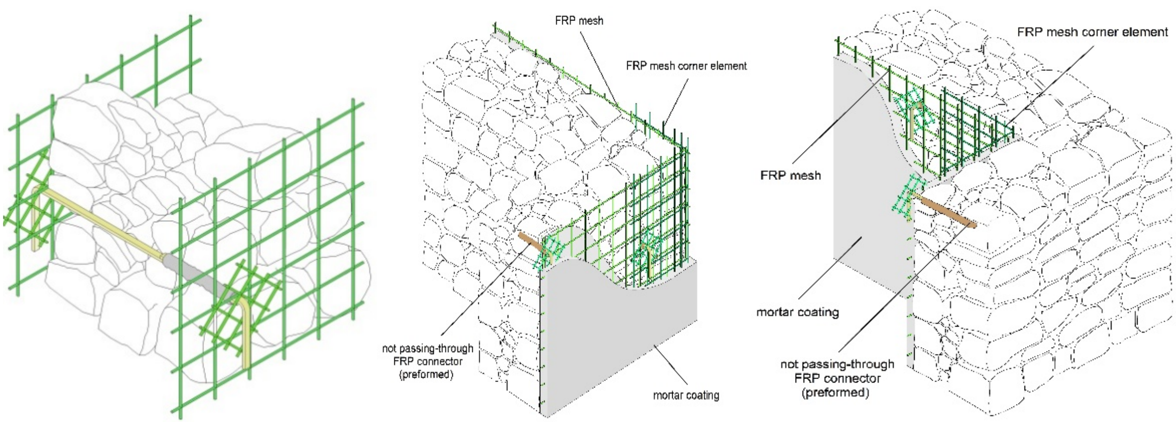


Fig. 4. Fibre Net system.

Material tests gave the results reported in Table 1. Building 1 was just repaired in order to eliminate damage that could affect the behaviour under horizontal forces. The interventions interested particularly the crack present under the window in the north elevation, and on some holes in the west elevation that were closed, and on the portion of internal masonry wall where the steel beams of the floor of the stairwell were removed.

Table 1. Main results of the preliminary tests on the mortar.

| Element | Compression strength (*MPa*) | Flexural strength (*MPa*) | Compression elasticity modulus (*GPa*) |
|---|---|---|---|
| Structural coating mortar cylinders | 11.82 | - | 9.76 |
| Structural coating mortar prisms | 14.55 | 5.28 | - |
| Cubes of coating (150 mm) | 13.44 | - | - |

## 3. Dynamic characterization tests

The dynamic characterization tests of the two buildings were performed using six Sara SL06 three-axial velocimeters arranged at the edges of the three levels called first, second and top, respectively. In the following, the two fronts indicated in the figures will be indicated respectively with "road" and "valley" and as longitudinal the sensors parallel to the longer side and transversal those parallel to the shorter side. Fig. 5 shows the position of the sensors in the buildings. The characterization was performed using both environmental noise, which included the effects of construction site activities, and impulses caused by the fall of a concrete cylinder weighing about half a ton from varying heights, up to about 5 m, as the source of stress. Spectral analysis of the recorded signals was performed, both for environmental noise and for impulse tests, calculating the power spectra density (PSD) for all sensors and the cross spectral density (CSD) between significant couples of sensors.


Fig. 5. Dynamic characterization: sensor deployment for each building and tests with mass drop.

The analysis of the PSDs and CDSs of the environmental noise test pointed out the resonance frequencies shown in Table 2. The impulse tests gave similar results in terms of modal shapes, with values of the resonance frequencies a little lower, because of a nonlinear behavior generated by a greater energy input. In order to evaluate the damping, the recording obtained at the top in the two orthogonal directions were filtered in small interval containing the resonance frequencies. The damping was estimated by means of the logarithmic decrease method. For building 1, the value of 3% was obtained in the transversal direction, while it was less 5.9% in the longitudinal direction. For building 2 these were to 5.3% and 4.5%, respectively.

## 4. Push in real analysis

To apply the thrust to both building 1 and building 2, a suitable contrast steel truss structure was realized on purpose (Fig. 6). A prism-shaped truss structure was used to apply the forces at four points of each building, at the intersections between the longitudinal walls and the second and third floor, respectively. Two hydraulic jacks were between the contrast structure and the prism-shaped one. The geometry was studied in order to have the upper forces double than the lower ones, simulating the first modal shape of each building.

The same system was used for both building 1 and building 2. It was fixed on a reinforced concrete plate, realized for each building and founded on four piles (length = 13 *m*, diameter 800 *mm*). The design of this structure and the foundation was performed by setting up a suitable finite element model, using SAP 2000 code, on the basis of the expected resistance of each building, suitably amplified. Pushover tests were carried out until collapse, increasing the horizontal forces at the two floors. In the tests, numerous potentiometer transducers were used to survey the displacements at several points of the specimen, to evaluate: i) floor drifts, ii) deformation in compression and in tension of the piers and the spandrels of the shear walls, iii) uplift, iv) sliding between foundations and shear walls, in the areas immediately below the points of application of the loads. During the test the graph of the applied force against the drift and the diagonal displacement of the shear walls, single piers and spandrels were monitored so to check the development of the test and to evidence the occurrence of some local damages (cracks, dislocations, etc).

Table 2. Experimental resonance frequencies (Hz) from ambient vibrations.

| Modal shape | Building 1 | Building 2 |
|---|---|---|
| Flexural, transversal direction | 6.05 | 7.43 |
| Flexural, transversal direction | 8.18 | 10.0 |
| Flexural, longitudinal direction | 9.88 | 12.3 |
| Torsion | 11.90 | 15.7 |

The experimental results were analyzed carefully and compared with those obtained from a suitable numerical model. This was developed adopting the force-based equivalent frame procedure already proposed to analyze the masonry wall in-plane structural response under lateral loadings (Addessi et al. 2015). The used macro-element formulation consists of the arrangement in series of a central elastic beam, two nonlinear flexural hinges at the ends and a nonlinear shear link, all characterized by a rigid-plastic response (Sangirardi et al. 2019). The model proved to be computationally efficient and suitable to interpret very well masonry experimental behavior. In Fig. 7 the force-displacement response curves obtained for building 2 are also shown. The numerical results (blue curve) are in good agreement with the experimental outcomes (red curve), giving a perfect match in the initial linear and nonlinear stage. Also, a good estimate of the limit load is numerically evaluated. The subsequent sudden drops of the numerical curves are related to the ultimate displacement values attained by some plastic hinges and correspond to the formation of severe nonlinear flexural/shear mechanisms.

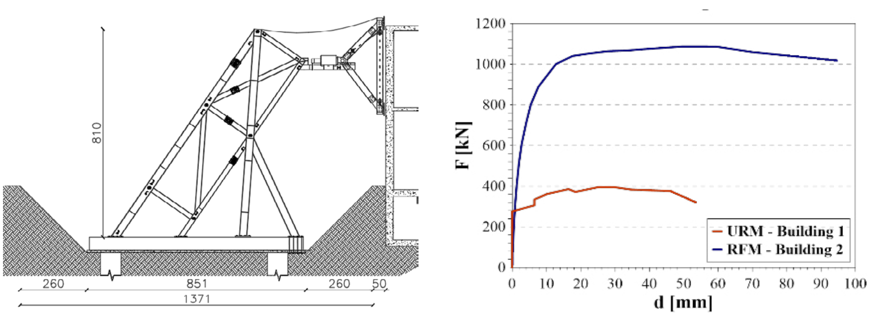


Fig. 6. Scheme of the push structure and experimental push-over diagram.

## 3 Conclusions

The realization and the main results of an experimental pushover carried on two very similar masonry buildings obtained from an existing two-story brickwork building, were shown. One building was simply repaired, by repointing the cracks caused by the earthquakes, the other was seismically strengthened using Fibre Net reinforced plaster or composite reinforced mortar. The plaster was applied on both faces of the load-bearing walls of the first level and only on the external face at the second level. The pushover tests, carried out until collapse on both buildings, using a steel structure placed on a reinforced concrete plate supported by four piles, pointed out a significant increases of the strength. The numerical analyses, performed by adopting a force-based equivalent frame procedure allowed to interpret very well the experimental behavior.

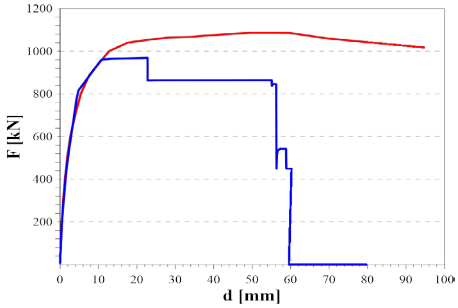


Fig. 7. Experimental (red) and numerical (blue) force-displacement response curve for building 2.

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