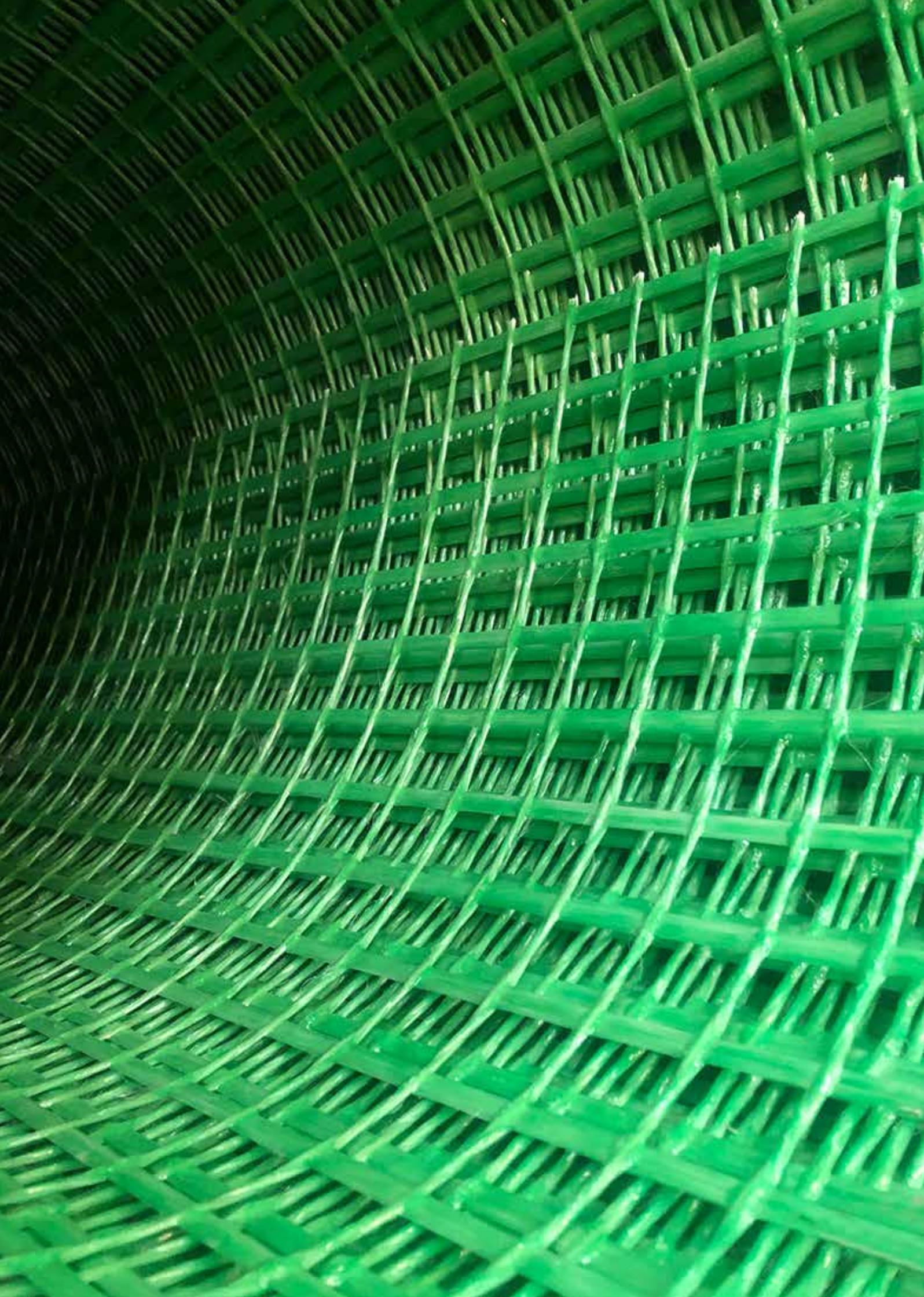# TECHNICAL MANUAL
RI-STRUTTURA
CRM SYSTEM

TECHNICAL MANUAL

RI-STRUTTURA

CRM SYSTEM

The "RI-STRUTTURA Technical Manual" aims to be a useful and practical working tool for professionals and companies operating in the construction sector.

It is the result of over twenty years of research and experimentation, as well as on-site experiences that have established Fibre Net as the leading figure in the field of composite materials applied to construction.

INDEX

1
INTRODUCTION

The CRM RI-STRUTTURA System, or Composite Reinforced Mortar, has been developed for the consolidation of existing buildings, drawing on the evolution of the traditional reinforced plaster technique widely used in structural reinforcement interventions following the Friuli earthquake of 1976.

Fibre Net has dedicated significant resources to developing a system that ensures effectiveness on all fronts, both mechanically and in terms of durability.

The latter is a fundamental characteristic to ensure the maintenance of performance over time, guaranteeing a prolonged service life of the structure. Particular attention has also been given to the need for easy use by construction workers on site and for quick verification by professionals.

The purpose of this document is to provide an executive and design approach for both professionals and site operators.

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2

PREMISES AND GENESIS OF RI-STRUTTURA FIBRE NET'S CRM SYSTEM

The journey that led to the genesis and improvement of the CRM System by Fibre Net and the various working groups that succeeded over the years has been challenging and complex, with numerous modifications along the way. This path began in 2001 with the first mechanical and durability tests carried out at the University of Salento. The monolithic FRP mesh, in its initial version, was made with standard glass fibers and isophthalic resins, but the outcome of the laboratory tests in aggressive environments highlighted insufficient long-term mechanical performance of the composite material. For this reason, in 2004, still in collaboration with the University of Salento, a shift towards the use of bisphenolic resins was decided, but even in this case, the durability tests revealed rather deficient behavior in aggressive environments. It was only in 2008, with the introduction of new types of glass fibers - the so- called Alkali-Resistant (AR) fibers - and with the first experimental campaigns of reinforcement applied to masonry, in collaboration with the University of Trieste - Prof. Gattesco, that the combination of highly durable fibers and vinyl ester resins achieved excellent results both in terms of mechanical performance and durability even in aggressive environments. Having defined the optimal combination of fibers and resins, this approach was extended to the creation of a connecting element between reinforcement and substrate, namely the connector, which underwent several considerations and modifications compared to the initial versions developed.

From the close contact with the construction industry and the mutual need to optimize the installation processes to provide a set of competitive products, several solutions and improvements were developed, including the modification of the connector from its original "U" shape to the current "L" shape, which is still maintained. In-house production has always been the driving force behind product improvement: meshes made with different materials and combinations, preformed angular elements with different geometries, new types of connectors are just some of the prototypes that, once industrialized, have been widely used in the world of structural reinforcement. The ever-growing evolution in the research sector, where research is considered a necessary tool to guarantee the credibility of an innovative system to a demanding audience and a regulatory framework not very inclined to change, such as that of structural designers, led us to face further experimental campaigns in 2012-2013, together with Prof. Gattesco (University of Trieste), Prof. Borri (University of Perugia), and their research groups. The company has constantly improved and enhanced the production process of CRM System components over the years, modifying the machinery and production lines.

A note on this: the production machinery for all CRM System components is developed directly by Fibre Net, based on ideas, creativity, and design applied to large- scale industrial production.

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The experimental campaigns carried out internally over the years, along with others undertaken with the University of Salento, have allowed us to further investigate the mechanical behavior of the applied System and that of durability, and to create innovative artificial aging protocols that have become a point of reference for the academic, institutional and industrial world.

These protocols have been introduced in the Slovenian Technical Approval, the European Technical Approval, as well as incorporated within the Italian Qualification Guideline of the CRM System.

The experiences gained by the company's in-house technical staff over the past 20 years, together with cross-skills in the structural, chemical and mechanical fields, have enabled important in-process modifications or real solutions to problems that arose during the experimental campaigns. These skills, combined with in-house chemical and mechanical laboratory facilities, have allowed us, over the 20 years of the company's existence, to constantly improve products and create new ones, such as to complement those already made, in a set of effective components such as those of the CRM System.

3

THE RI-STRUTTURA SYSTEM
FEATURES AND FIELDS OF APPLICATION

The RI-STRUTTURA System employs the "reinforced plaster" technique applied to one or both faces of the masonry using GFRP (Glass Fiber Reinforced Polymer) meshes, connectors, and angles, combined with preferably lime- based mortars. This method creates reinforced plasters with reduced thicknesses (about 3 cm) capable of enhancing the mechanical performance of masonry walls.

The GFRP mesh, comprised of long glass fibers impregnated with thermosetting resin, is produced using "TextrusionR" technology. In the formation of the mesh, fibers from two directions are woven orthogonally to create a substantially monolithic grid.

CHARACTERISTICS

HIGH MECHANICAL STRENGTH

LIGHTWEIGHT AND LOW PROFILE SYSTEM

HIGH CORROSION RESISTANCE

COMPATIBLE WITH MORTARS OF DIFFERENT TYPES

NON-MAGNETIC AND RADIO-TRANSPARENT

NON-CONDUCTIVE OF ELECTRICITY

ADVANTAGES

DURABILITY AND EFFECTIVENESS OF THE INTERVENTION

WIDESPREAD AND UNIFORM MECHANICAL IMPROVEMENT

BREATHABILITY OF THE MASONRY AND REVERSIBILITY

EASE AND SPEED OF APPLICATION, SITE SAFETY

REDUCTION OF COSTS AND INSTALLATION TIME

REDUCTION OF THERMAL BRIDGES AT CONNECTION POINTS

3.1   SYSTEM COMPONENTS
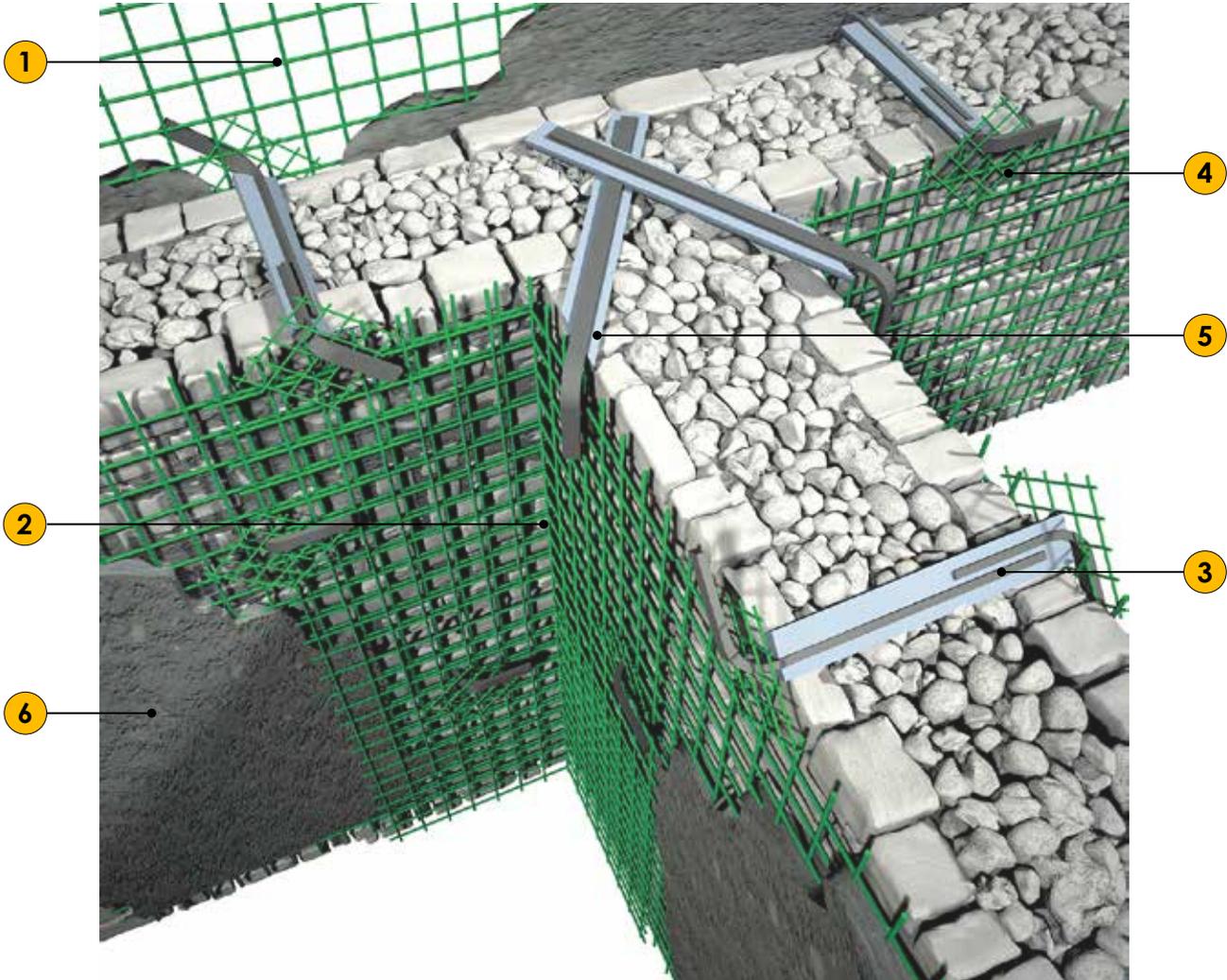
1 PREFORMED MESHES

Preformed mesh in GFRP of various weights.
Mesh size: 33×33/66×66/99×99 mm

2 PREFORMED ANGLES

Preformed right-angle GFRP element of various weights.
Mesh size: 33×33/66×66/99×99 mm

3 PREFORMED CONNECTORS

Preformed "L" shaped GFRP connector for connecting the mesh to the masonry.
Long side length: from 10 to 100 cm

4 PREFORMED MESH RETENTION PANELS

Preformed GFRP mesh retention panels.
Mesh size: 33×33 mm

5 CHEMICAL ANCHOR

Bi-component chemical anchor for fixing connectors

6 STRUCTURAL MORTARS

MATERIA RINFORZA: Pre-mixed mortar based on hydrated lime and hydraulic binders. EPOCA CALCE: pre-mixed mortar based on natural hydraulic lime NHL 3.5.NHL 3,5

3.2   MASONRY REINFORCEMENT

The intervention with the RI-STRUTTURA System involves the application of a thin layer (about 3 cm) of pre-mixed mortar suitable for structural applications, compatible with the system, reinforced with preformed GFRP meshes and accessories, on one or both faces of the structure.

This intervention allows for a homogeneous and widespread structural improvement, an increase in shear strength in-plane, and flexural strength in-plane and out- of-plane with a modest increase in structural stiffness.

3.3   STRENGTHENING OF MASONRY COLUMNS

The RI-STRUTTURA System can also be used for the reinforcement of masonry columns. This intervention is effective for repairing damaged or deteriorated elements and for reinforcing intact elements in view of a static or seismic upgrade of the structure. The connection of the reinforcement system is achieved by applying "L" shaped elements in GFRP, typically at

a rate of 4 to 6 units per square meter staggered. It is recommended to use GFRP bars to connect the pillar to the floor slabs or to the foundation structure, thus ensuring continuity in terms of tensile performance of the entire CRM System and GFRP bars.

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3.4 REINFORCEMENT OF MASONRY VAULTS AND VAULTED CEILINGS

The RI-STRUTTURA intervention on arches and vaults involves the construction of a thin mortar cap on the extrados and possibly also on the intrados, preferably based on lime, reinforced with a preformed GFRP mesh. The connection to the vault is then ensured by transverse "L" shaped connectors, also made of GFRP. This intervention allows for a homogeneous and widespread structural improvement, with high mechanical and ductility characteristics, but with a modest increase in structural stiffness. In the case of vaults with frescoes on the intrados, it is recommended to use a reinforcement/support connection by gluing stainless steel plates to the extrados, which allow for the transfer of shear forces between the vault and the GFRP mesh.

3.5 REINFORCEMENT OF FLOOR SLABS

The RI-STRUTTURA intervention on floor slabs involves the construction of a reinforced slab on the extrados, using preformed GFRP meshes and properly connected to the underlying slab through the use of metallic connectors. The RI-STRUTTURA System allows for increasing the resistance of the floor slab to both gravitational actions and horizontal actions such as seismic and wind loads, enabling the creation of a composite slab that is sufficiently rigid to distribute horizontal seismic actions and where reinforcement helps distribute the loads acting on the slab itself. For better seismic behavior of the building, it is necessary to connect the floor slab to the masonry walls using preformed GFRP bars.

3.6 MASONRY COMPATIBILITY OF THE REINFORCEMENT APPLICATION

The use of cementitious plasters reinforced with welded wire meshes has been found to be harmful to historic buildings, as it brings about significant increases in stiffness due to the high elastic moduli of the concrete used. This leads to cracking patterns with detachments and spalling caused by the expansion of the reinforcements due to corrosion, as well as chemical and physical incompatibility between the masonry substrates and Portland cement.

As a consequence, within the latest "Directive of the President of the Council of Ministers for the assessment and reduction of seismic risk to cultural heritage," masonry consolidation interventions using reinforced plaster with welded wire meshes are defined as "invasive and not consistent with conservation principles."

The CRM reinforcement system described in this technical handbook proposes the replacement of metal mesh with preformed GFRP meshes with high durability. The rigid and chemically damaging cementitious mortar can be replaced by more ductile mortars completely free from cement, produced with hydraulic lime or natural hydraulic lime NHL.

Thanks to the chemical inertness of the GFRP reinforcements, the problem of reinforcement corrosion is completely eliminated, and it is possible to reduce the thickness of the protective plaster layer. In particular, in the case of non-through holes made in mortar joints, the RI-STRUTTURA CRM System is non-invasive, compatible with original materials, and reversible.

3.7 REVERSIBILITY OF THE APPLICATION

To demonstrate the low invasiveness of the reinforcement application, a reversibility test was carried out on a historic tuff masonry building located in the center of Naples near the Monastery of Santa Chiara [1]. Three wall panels were identified on which a lime-based mortar plaster approximately 30 mm thick, reinforced with GFRP mesh with mesh dimensions of 66x66 mm, was applied.

The three samples differed in the characteristics of the mortars used:

▪ NHL mortar with compressive strength of 8 MPa;

▪ NHL mortar with compressive strength of 13 MPa;

▪ NHL mortar with pozzolan with compressive strength of 15 MPa.

Once the plaster had matured, it was removed using a mechanical breaker. Compared to removing traditional reinforced concrete, the operation was fast and effective for all three prepared panels. The lower stiffness and strength of the mortar allowed for easier detachment from the masonry support without significantly damaging the masonry.

The behavior given by the nature of the material (glass fiber and thermosetting resin) comprising the CRM reinforcement element allows for simple and fast cutting and removal without interfering with the masonry texture. The only trace, generally negligible, is that left by the transverse connector, which will be cut flush with the masonry.

3.8　INSTALLATION METHODS ON MASONRY

The application of the RI-STRUTTURA CRM System for reinforcing masonry walls on both faces of the wall surface is carried out in the following stages:

1 Studying the masonry, in terms of thickness typology and materials analysis.

2 Removal of the existing plaster and mortar from the joints between the masonry elements (10-15 mm deep), from both faces of the panel, and optionally rebuilding missing or particularly damaged masonry parts.

3 Washing and saturating the surface, followed by the application of a first layer of structural mortar render (if necessary, in the case of small mesh sizes that may make it difficult for the mortar to pass through), drilling holes using a rotating drill, and applying the GFRP mesh on one side, ensuring it does not adhere to the wall.

4 Cleaning of the hole using a high-pressure air jet and insertion of the GFRP connector with a length equal to the thickness cf the masonry.

5 Installation of the GFRP mesh on the opposite side of the masonry surface, ensuring that it is not adherent to the masonry; if necessary, cutting of the excess part of the GFRP connector, using diamond discs for brickwork.

6 Insertion of the second GFRP connector into the hole, creating an overlap of at least 10-15 cm.

7 Injection of epoxy resin to bond the connectors together; if necessary, application of distribution mesh retention paneles.
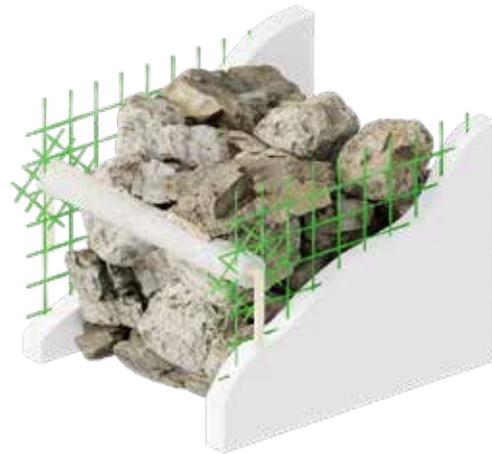
8 Application of a single layer of structural mortar while keeping the mesh centered. The plaster has a thickness of approximately 30 mm on each side.

NOTES:

▪ In areas where connectors overlap, the hole should have a diameter of at least 18 mm, while a 12 mm diameter hole is sufficient where the connector is single;

▪ The installation of the mesh can be performed by unrolling the roll from bottom to top or vice versa, between the scaffolding and the masonry;
▪ To ensure mechanical continuity, the mesh should overlap for approximately 15-20 cm;

The installation operations of the system must be carried out by experienced personnel and must comply with the instructions provided in the technical datasheet of the individual products.

3.9　INSTALLATION METHODS ON FLOOR SLABS
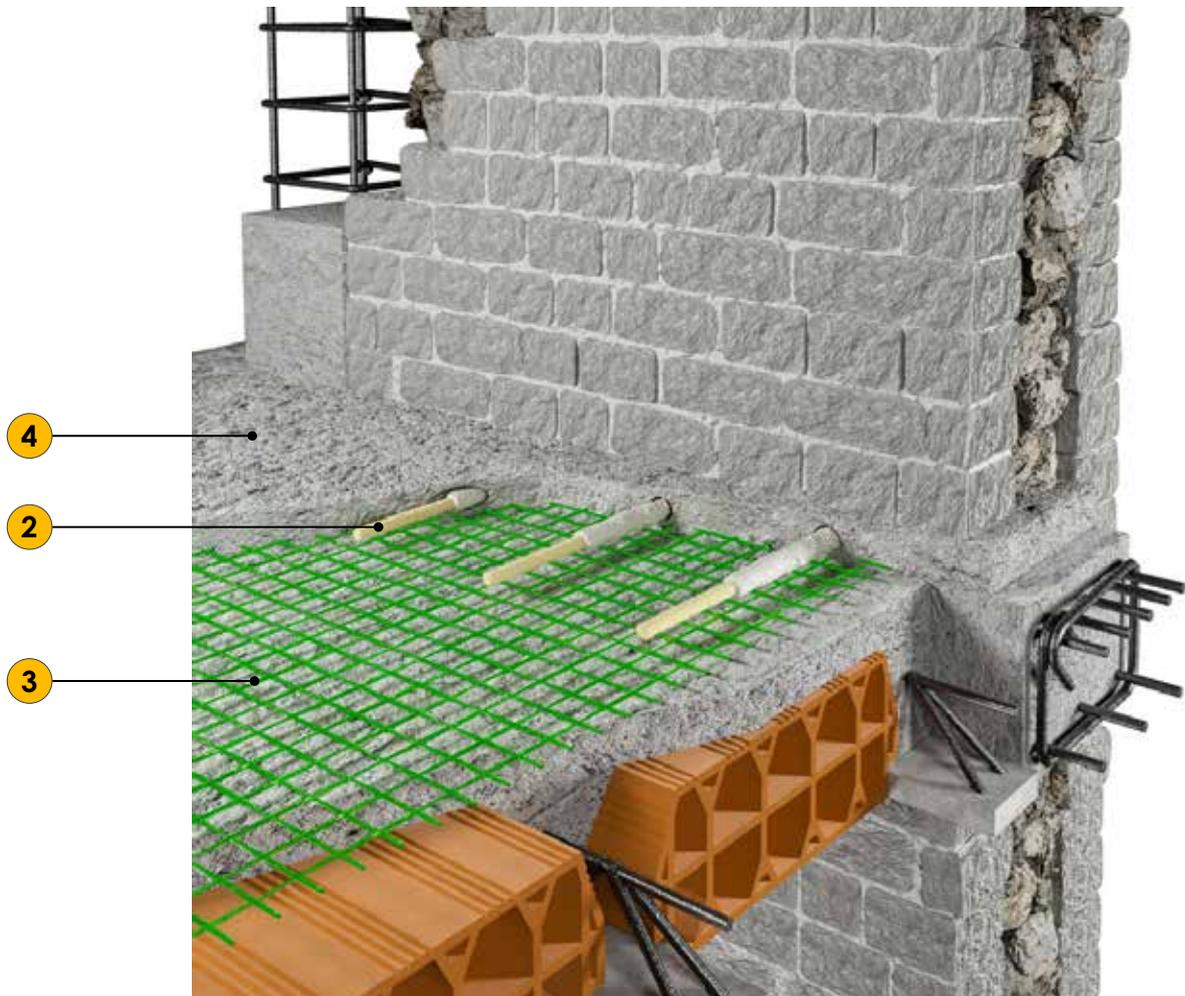
1

Analysis of the floor slab, material analysis, and selection of the type and arrangement of connectors required. Analysis of the masonry, selection of the type and arrangement of elements for perimeter anchorage.

2

Application and fastening of the connectors if provided, according to the designer's instructions.

3

Installation of the GFRP mesh on the extrados of the floor, with an overlap of at least 15 cm;
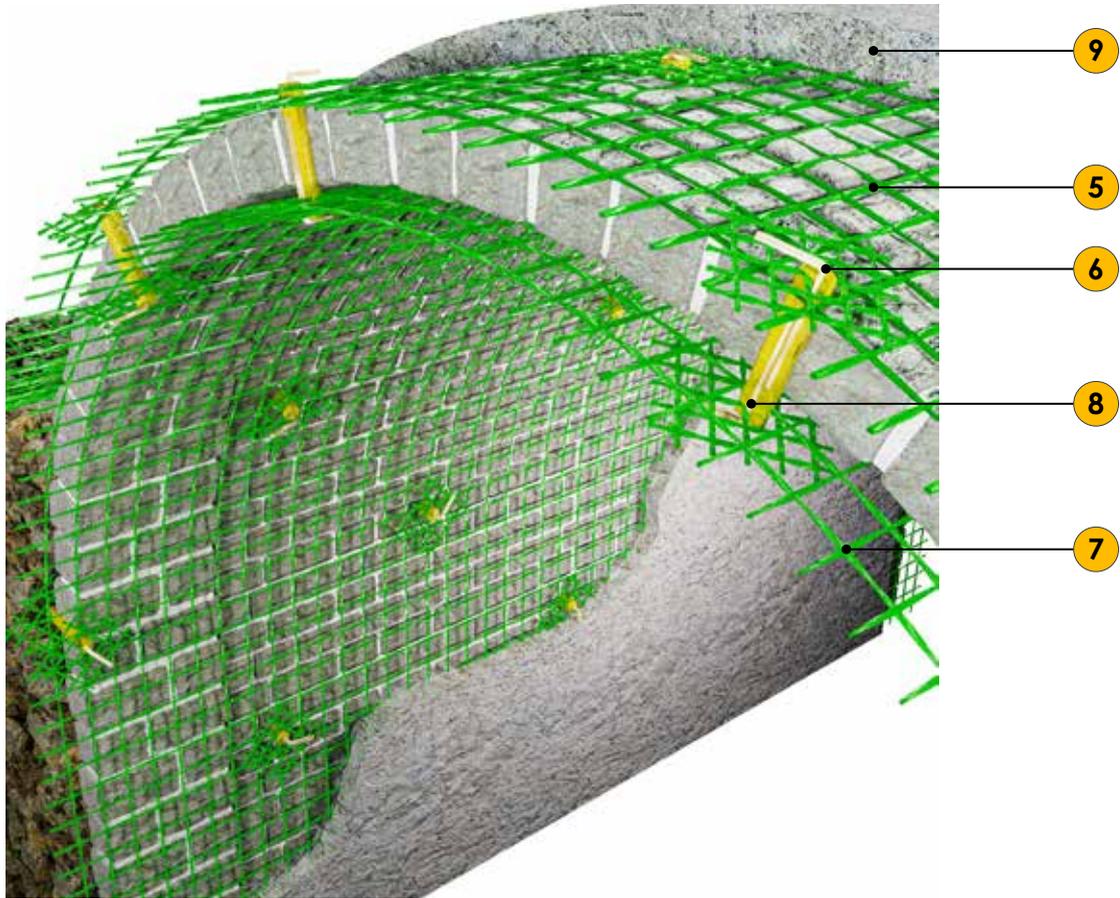
4

Pouring of concrete as per the design thickness (approximately 20/40 mm).

3.10 INSTALLATION METHODS ON MASONRY VAULTS

1

Removal of the existing plaster and damaged parts, scarification of approximately 10/15 mm of the bedding joints to promote the adhesion of the mortar (on the extrados and/or intrados depending on the intervention possibilities);

2

Cleaning of the vault with compressed air, washing, and saturating the surface with water (where possible), followed by the application of a first layer of mortar;

3

Drilling of holes with a diameter of 24 mm for the pass-through connectors in the number specified by the project, preferably using rotating tools. A hole with a diameter of 14/18 mm is sufficient where only one connector is expected (non-pass-through holes or reinforcement on only one side of the vault);

4

Cleaning of the holes with compressed air;

5

Arrangement of the mesh on one side of the vault. Cutting of the mesh is carried out using shears and/or construction nippers or with an angle grinder. Overlap the mesh strips by approximately 15 cm to ensure mechanical continuity. Do not bend the mesh at sharp angles to avoid potential fiber breakage;

6

Insertion of the GFRP connector with a length equal to the thickness of the vault (or less in the case of non-pass-through connections). If necessary, cut off the excess part of the GFRP connector;

7

Possible installation of the GFRP mesh on the opposite side of the vault;

8

If applicable, insertion of the second GFRP connector into the hole, creating an overlap of at least 10/15 cm between the two connectors, and injection of epoxy resin to bond the two elements together. If required, application of distribution mesh retention panels;

9

Application of the mortar plaster with project specifications, with a minimum thickness of approximately 30 mm, on the extrados and/or intrados. To prevent cracking of the plaster mortar, the GFRP elements must be covered with at least 1 cm of mortar. The mesh should be positioned midway within the mortar thickness.

3.11 INSTALLATION METHOD ON LEAF VAULTS

1
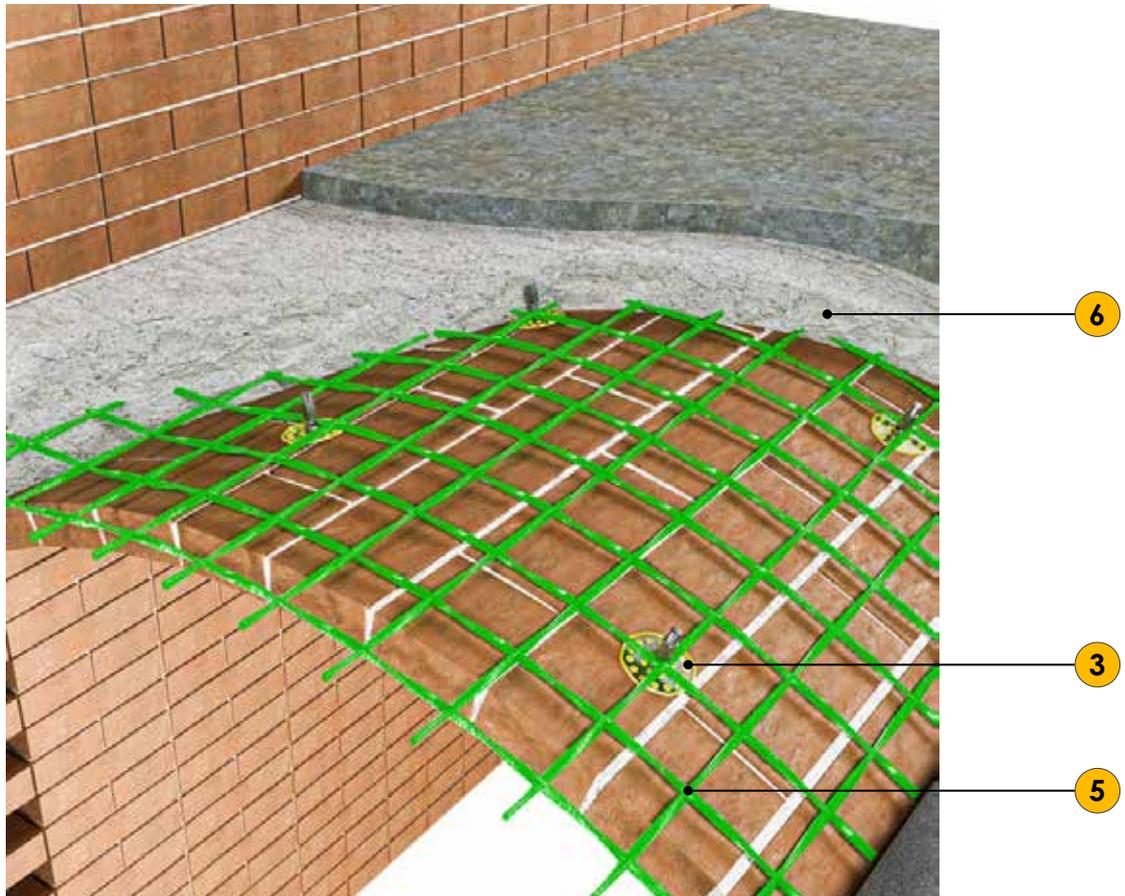
Removal of the existing plaster. Cleaning of the connector application surface with a wire brush followed by overall cleaning of the vault with compressed air;

2

Spot application of epoxy resin on the surface of the vault for the subsequent application of the sleeve;

3

Placement of the stainless steel AISI 316 sleeve on the freshly applied resin, applying pressure until the resin emerges from the base of the plate;

4

Wait for the resin to polymerize (approximately 24 hours);

5

Arrange the mesh on the vault, overlapping the mesh strips by approximately 15 cm to ensure mechanical continuity;

6

Apply the mortar plaster, with a minimum thickness of 3 cm. The mesh should be positioned in the middle of the mortar thickness.

3.12 STITCHING WITH GFRP BARS

The anchoring between the intersections of masonry can be improved by inserting GFRP reinforcement bars, of suitable diameter, inclined at 45° relative to the horizontal orientation of the panels to create a reinforced grid capable of absorbing the tensile forces induced by external actions.
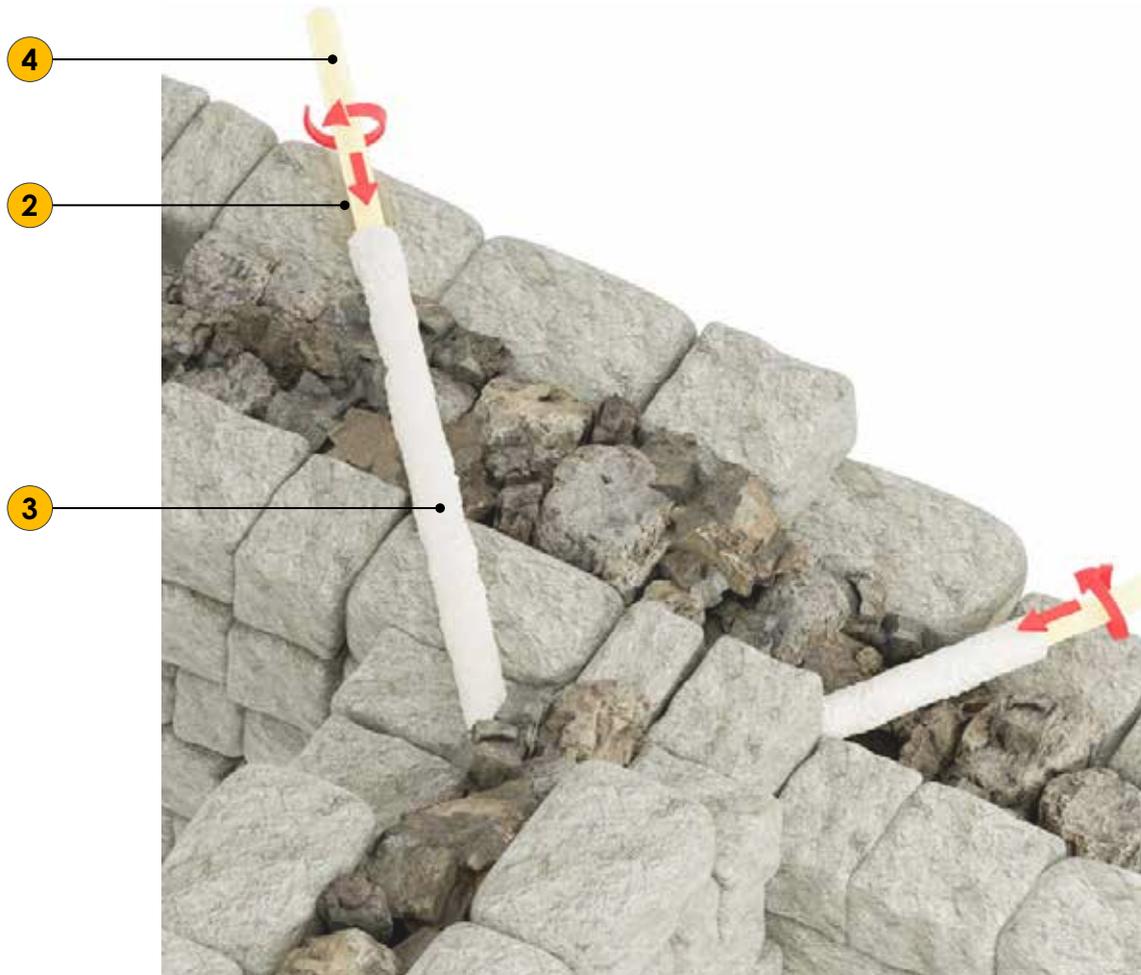
INSTALLATION METHOD FOR GFRP BARS

1

Study of the masonry and identification of the hole arrangement;

2

Drilling of the hole as per project specifications using a rotating tool. Cleaning of the hole by means of compressed air blowing and washing with water to ensure better adhesion;

3

Injection of resin or lime mortar;

4

Insertion of the GFRP bar of suitable length into the hole, rotating it around its axis to ensure a perfect distribution of resin or mortar around the bar.

4

RESEARCH AND DEVELOPMENT: EXPERIMENTAL INVESTIGATIONS AND THE BUILDING-BLOCK APPROACH

Experimental tests are essential to provide evidence of the behavior of a consolidation system and its role in the structural response of masonry elements and structures. To optimize resources and evaluate the effectiveness of the CRM System, over the past decade, the so-called Building-Block Approach or modular approach has been applied **[2], [3]**: it is a systematic process consisting of a series of tests of increasing complexity, ranging from individual components of the system to entire reinforced buildings (**Fig.01**).

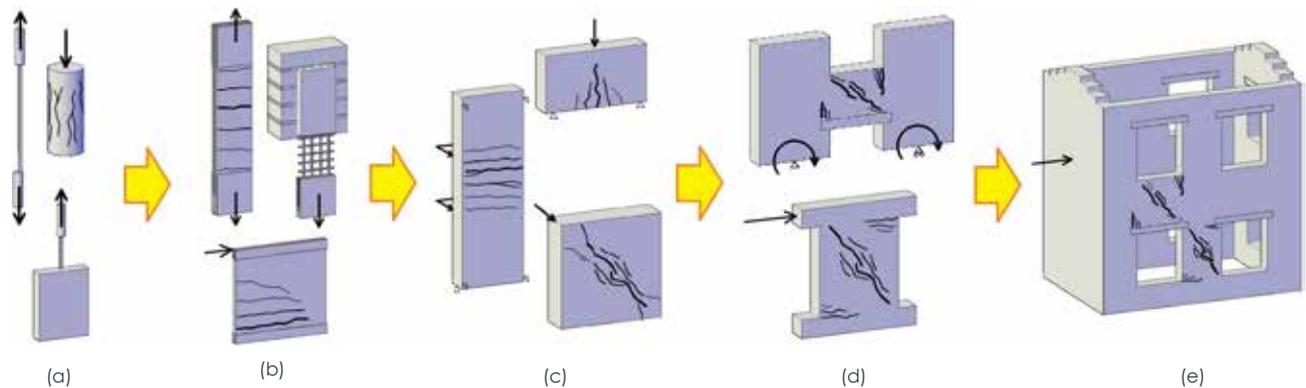(a)   (b)   (c)   (d)   (e)

Fig. 01 - *The Building-Block Approach applied for the experimental study of the CRM technique at the level of (a) single material/interface, (b) CRM specimen, (c) masonry elementary sample, (d) structural element, and (e) building.*

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4.1 THE EARLY EXPERIMENTS AND INITIAL CONFIRMATIONS OF AN EFFECTIVE CONSOLIDATION SYSTEM

The CRM System was conceived, from its inception, to offer existing constructions an innovative consolidation system by improving traditional techniques, using the mechanical principles of reinforced plaster. This technique had been widely used in consolidation interventions following the Friuli Earthquake of 1976, but the aim was to renew it by selecting materials to ensure effectiveness in all its aspects, from mechanical to durability, a fundamental aspect for ensuring the long-term maintenance (service life) of the intervention's mechanical performance.

Having identified the optimal combination of raw materials for the production of CRM System components, the focus shifted to the initial experimental tests of the System applied to masonry: diagonal compression tests and complementary tests for the mechanical characterization of the System and its components were chosen as they represented a good compromise between the results obtained and the costs of setting up and conducting the tests. For the initial evaluations of the effectiveness of the CRM System on various types of masonry, preliminary experimental characterization tests were carried out, particularly tensile tests and adhesion tests **[4]**: tensile tests provided the typical trilinear behavior of reinforced concrete elements: an initial non-cracked phase, governed by the mortar of the plaster, a phase of crack formation, and a phase of crack stabilization, governed by the characteristics of the GFRP mesh, ending with the rupture of the longitudinal fibers.
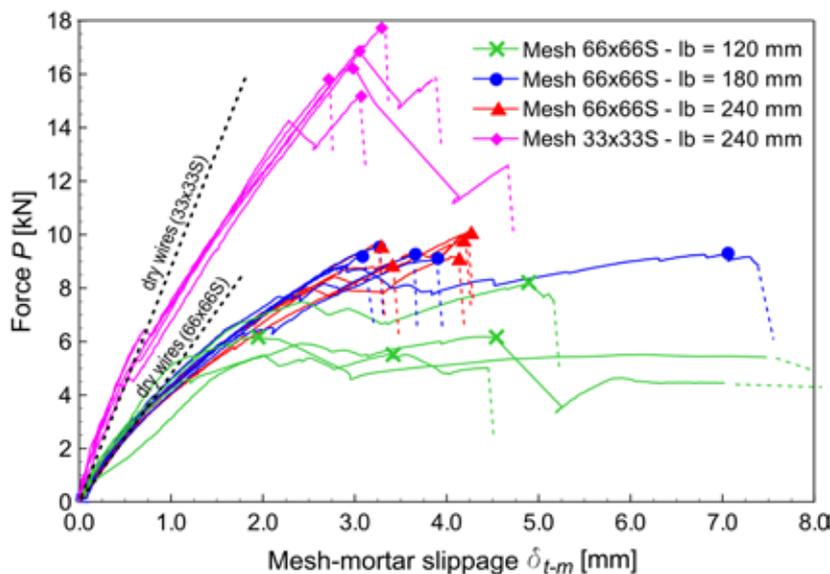

Fig.02 - *Characterization tests of CRM elements*

The cracks generally formed at the transverse threads; there was evidence of a stiffening effect of the threads between two cracks due to the intact mortar between the cracks (tension stiffening). Adhesion tests (**Fig.02**), on the other hand, were performed on samples of solid brick masonry to which a layer of CRM was applied on one face, considering different bond lengths. The masonry sample was fixed at one end of a tensile testing machine, using a steel device; the protruding GFRP mesh at the other end of the sample was fixed to the other end of the machine, allowing a tangential tension to be applied between the mortar and the masonry.

To avoid parasitic bending, the forces applied at both ends of the sample were rigorously aligned. The relative displacement between the excess mesh and the edge of the mortar coating was measured. The tests provided different results depending on the bond length: for shorter bond lengths, yielding was caused by sliding at the mesh-mortar interface; for longer bond lengths, longitudinal filament rupture was instead achieved. As shown in Fig. 02, the first mode of collapse (green curves) implies a less brittle behavior compared to the second (magenta curves).

4.1.1 THE INITIAL INVESTIGATIONS INTO THE IN-PLANE BEHAVIOR OF MASONRY INVOLVED DIAGONAL COMPRESSION TESTS.

More than 150 diagonal compression tests were conducted on masonry samples measuring 1160x1160 mm^2 **[5]**, **[6]**, **[7]**, **[8]**, **[9]**, **[10]**, **[25]** considering various types of masonry (**Fig.03**):

▪ Solid bricks, thickness 250 mm;

▪ Solid bricks, thickness 380 mm;

▪ Double-leaf solid bricks with intermediate filling consisting of a conglomerate formed by pieces of stones and very lean mortar, thickness 120+140+120 mm;

▪ Hollow clay blocks with large horizontal voids;

▪ Rough stones, thickness 400 mm;

▪ Rough stones, thickness 700 mm;

▪ Pebbles, thickness 400 mm.

Different types of hydraulic lime mortar were used for the masonry, while various mortars based on lime and mixed lime-cement were used for the plaster, thus expanding the experimental variables of the CRM system components.

Fig. 03 - Samples of different types of masonry subjected to diagonal compression: (a) Solid bricks with two headers 250 mm - S, (b) Solid bricks with three headers 380 mm - S2, (c) Double-leaf solid bricks with filling in pieces of stone bonded with lean mortar 120 + 140 + 120 mm - I, (d) Hollow clay blocks with large horizontal voids - H, (e-f) Rough stones 400 mm - R and R2, (g) Rough stones 700 mm - R3, and (h) Pebbles 400 mm - C.

During the application of the reinforced plaster, it was observed that in solid brick masonry, the thickness of the plaster is almost constant due to the planarity of the masonry surface; therefore, an average thickness of about 30 mm can be assumed.

Conversely, in stone masonry, where the faces are more irregular due to the geometry of the blocks, a higher average thickness was obtained, specifically 35 mm in samples made with rough stones and 45 mm in samples with pebbles.

Test apparatus and procedures

To facilitate testing operations and prevent damage to the masonry, the specimens were built on a wooden base with a removable part, allowing the test system to be installed without moving the specimens (**Fig.04**).
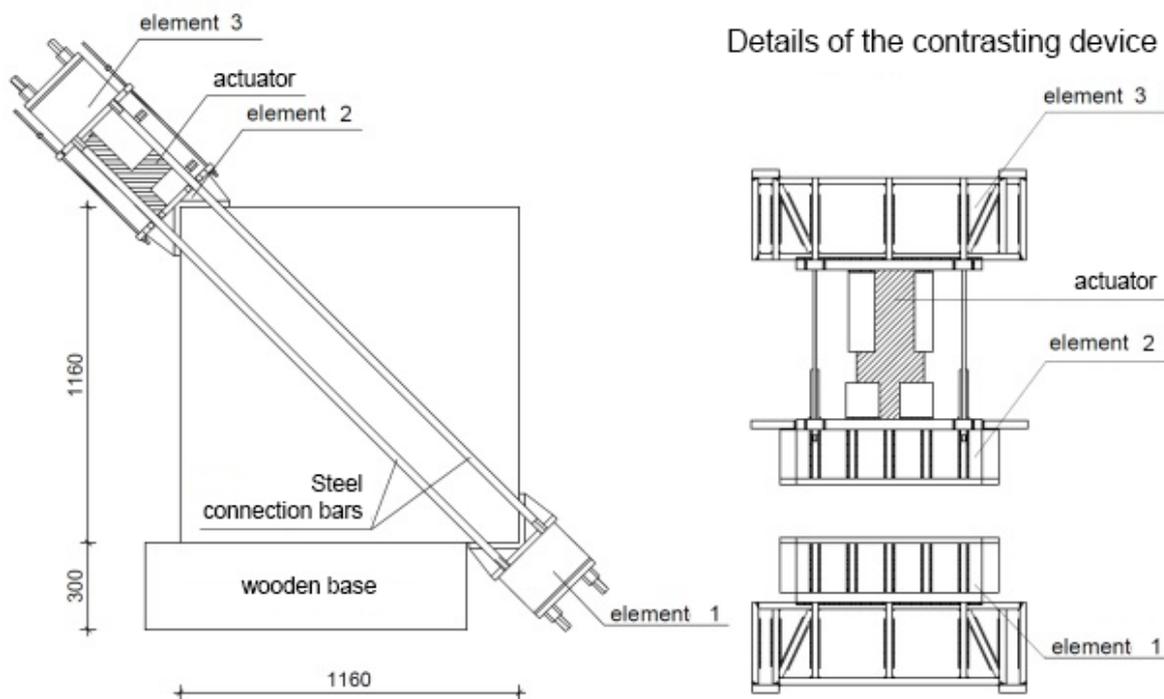
Fig. 04 - *Schematic representation of the setup for diagonal compression tests.*

The loading apparatus consists of two independent steel elements "1" and "2" arranged at two opposite edges with respect to a diagonal of the specimen. To limit the confining action by friction in the contact areas between the apparatus and the masonry, a PTFE (polytetrafluoroethylene) sheet was interposed. At the top of element "2," a double-acting hydraulic actuator, controlled by a manual pump, was installed and counteracted by a third steel element ("3") connected to element "1" via four steel bars (two on each side).

The loading procedure followed an increasing sequence of loading and unloading cycles until reaching the peak value of resistance. Then, the test continued by imposing cycles with increasing displacement targets until the specimen ruptured. The load was measured with a pressure transducer, while to detect displacements during the test on both sides of the specimen, pairs of potentiometric transducers were arranged along the diagonals, on a measuring base of approximately 1150 mm.

Main experimental results obtained

The unreinforced masonry samples in solid bricks and perforated bricks showed an approximately linear behavior up to the maximum load value; subsequently, there was a sudden drop in strength after the appearance of a stepped diagonal crack. In the double-headed brick samples, detachment of the brick walls from the lean conglomerate filling was also observed. In the unreinforced stone samples, although a reduction in stiffness was noted after the first cracking, the load continued to increase for a while. Later, the crack widened, and other cracks formed in the direction of the force with a gradual decrease in strength as deformation increased.

The irregular shape of the stone modules favored an interlocking effect, resisting crack opening. Depending on the size and nature of the stone elements, this effect was less pronounced in the stone and pebble masonry samples. In general, some differences were observed in the peak loads reached by samples with the same characteristics, probably due to minor differences during construction. However, it can be concluded that the higher the strength of the mortar, the higher the values of both the shear modulus and the strength. Additionally, similar maximum loads were reached for samples of two-headed and double-headed brick masonry,

indicating that the lean conglomerate filling did not provide any additional strength contribution but only a slightly more gradual strength decay. However, for some double-headed masonry, a significantly lower shear modulus (almost halved) was observed.

In the reinforced samples, a diagonal crack typically formed in the plaster diagonally just before reaching the peak force. With increasing deformation, other cracks gradually formed, almost parallel to the first one (**Fig.05**). These cracks also affected the masonry, following a similar pattern to that observed in unreinforced samples.

The GFRP mesh resisted crack opening by bearing tensile stresses, resulting in a gradual and moderate decrease in strength in the post-peak branch, overall achieving a highly ductile behavior of the masonry sample. At tensile strains εt of approximately 0.6-0.7%, the progressive rupture of some mesh wires began. Furthermore, as the stresses increased, an increase in damage to the plaster mortar between the cracks was observed, with significant local detachments, but without leading to the collapse of the structure.

(a-d) DIAGONAL COMPRESSION TESTS ON SOLID BRICK MASONRY S

(a)

(b)

(c)

(d)

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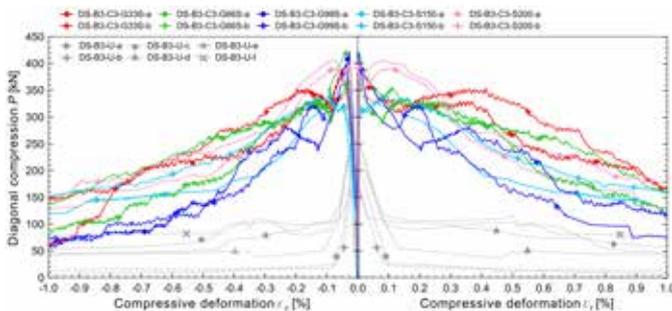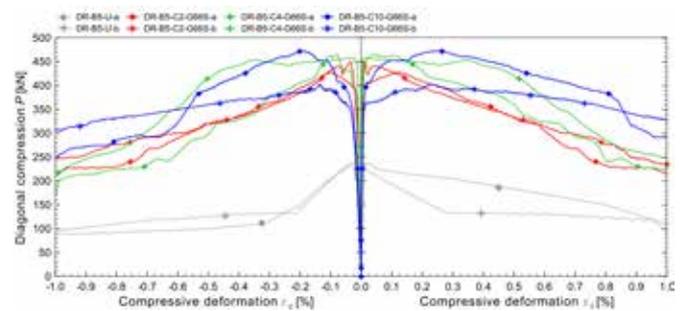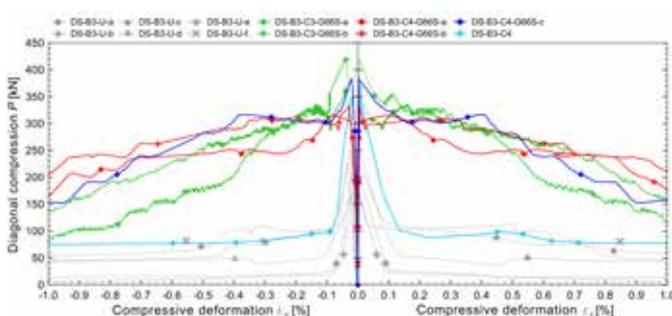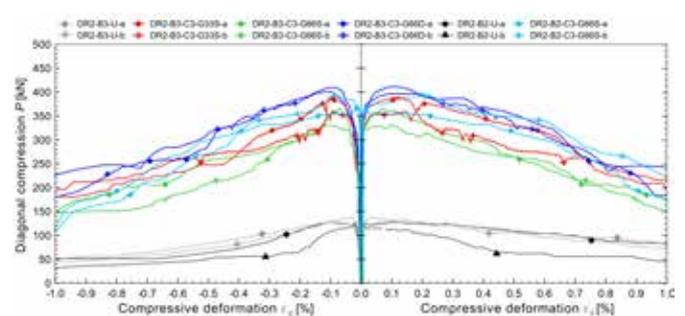
(e) DIAGONAL COMPRESSION TESTS ON SOLID BRICK MASONRY S2
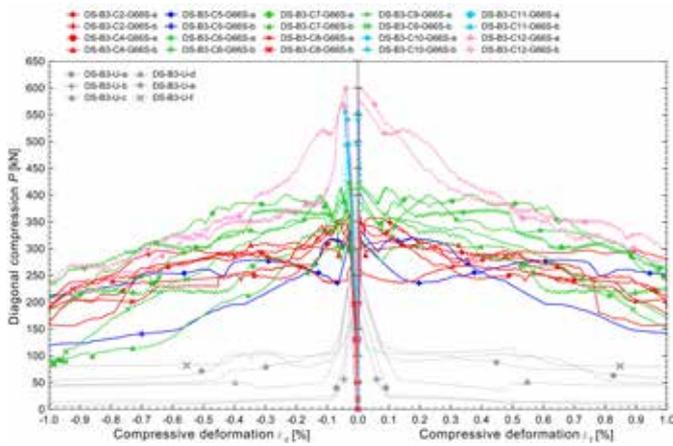
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(f) DIAGONAL COMPRESSION TESTS ON RUBBLE STONE MASONRY R
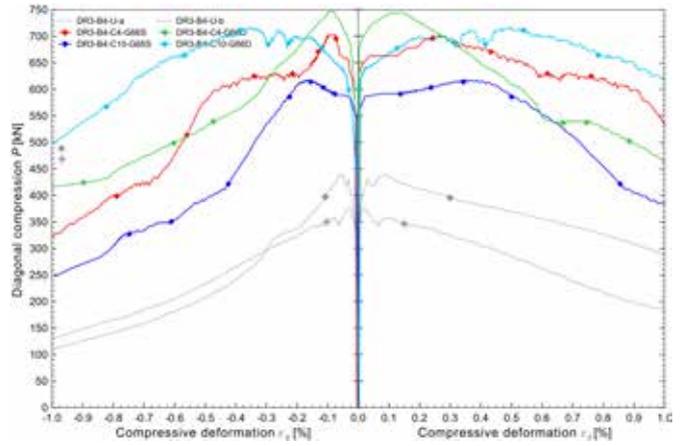
(f)

(g) DIAGONAL COMPRESSION TESTS ON RUBBLE STONE MASONRY R2

(g)
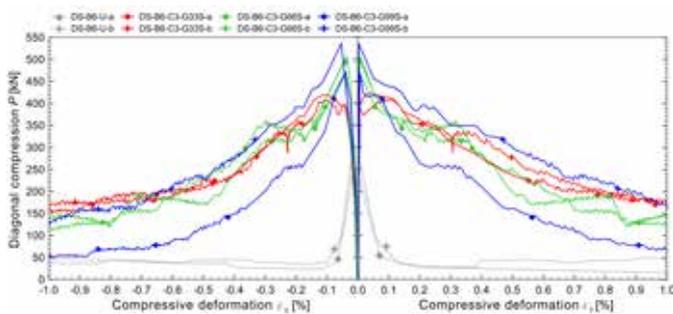
(h) DIAGONAL COMPRESSION TESTS ON RUBBLE STONE MASONRY R3
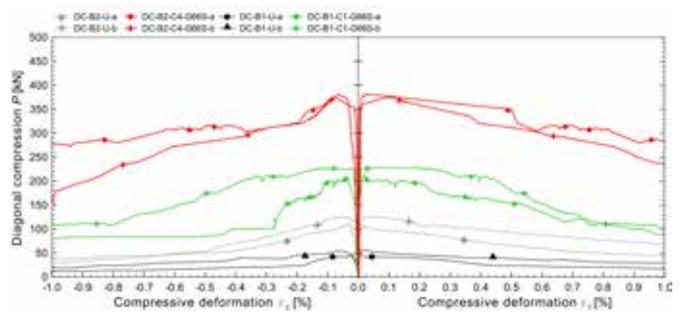
(h)

(i) DIAGONAL COMPRESSION TESTS ON RUBBLE STONE MASONRY WITH PEBBLES
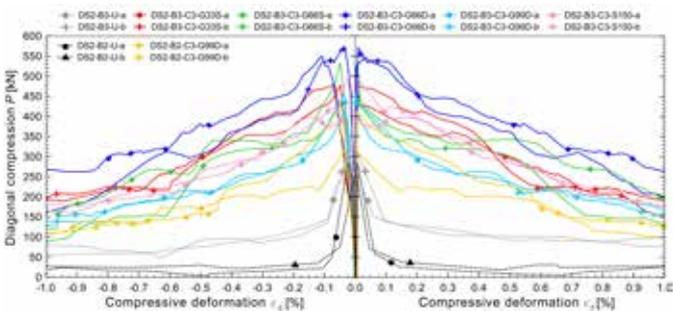
(i)

(j) DOUBLE-HEADER BRICK MASONRY, I, AND MASONRY IN BRICK BLOCKS WITH LARGE HORIZONTAL HOLES, H
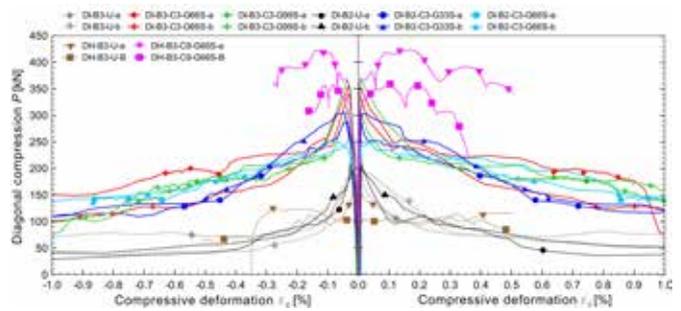
(j)

Fig. 05 - *Capacity curves of masonry samples subjected to diagonal compression tests: (a-d) solid brick S, (e) solid brick S2, (f) rubble stone R, (g) rubble stone R2, (h) rubble stone R3, (i) pebbles, and (j) double-header brick masonry I, and masonry in brick blocks with large horizontal holes, H.*

In general, a particular effectiveness of the CRM technique has emerged for weaker masonry: the peak strength was found to be limitedly influenced by the amount of reinforcement, due to the low geometric percentage and the low modulus ratio of the mesh compared to that of the plaster. However, the presence of GFRP mesh is crucial to ensure the ductility of the masonry panels. Its role is clearly highlighted in the brick samples, where there is a sharp drop in strength immediately after the peak and a limited number of cracks in the plaster for the sample without mesh, indicating a poorly ductile behavior of the unreinforced masonry.

A smaller mesh size of the mesh generally resulted in a greater spread of cracks, demonstrating the high diffusive capacity of the reinforcement. The variation in the number of connectors does not seem to appreciably influence the performance of the reinforced samples. However, in samples made of double-header masonry with filling in lean conglomerate, detachment of the brick facing was observed in tests of unreinforced samples, while it did not occur in reinforced samples, although significantly higher stress levels were reached. In general, it is important to highlight that the connectors play a fundamental role when there is also significant axial compression.

4.1.2 THE OUT-OF-PLANE BEHAVIOR OF MASONRY

To analyze the out-of-plane behavior of masonry reinforced with CRM, "four-point" vertical bending tests were conducted on full-scale masonry samples with a width of 1000 mm and a height of 3000 mm. Three different types of masonry were considered: solid brick masonry (250 mm thickness), rough-cut sandstone block masonry (400 mm thickness), and river pebble masonry (400 mm thickness, stone dimensions approximately 120/130 mm). The samples were constructed using bedding mortars based on natural hydraulic lime. Two samples were tested for each masonry type: one reinforced with the CRM system applied on both faces and one unreinforced, for comparison purposes.

Fig. 06 - Out-of-plane testing apparatus: (a) Overview and details (b) of the loading area and (c) of the base support.

In the samples reinforced with CRM, a progressive formation of multiple horizontal cracks was observed on the plaster slab subjected to tension, localized in the central third of the height, where the bending moment remains approximately constant.

Each crack formation was associated with a moderate reduction in load, which then resumed growing until the next crack formed. The collapse of the reinforced samples occurred when, at the location of the widest crack, the vertical wires of the mesh reached tensile failure **(Fig. 07)**, almost simultaneously, resulting in a sudden collapse of resistance
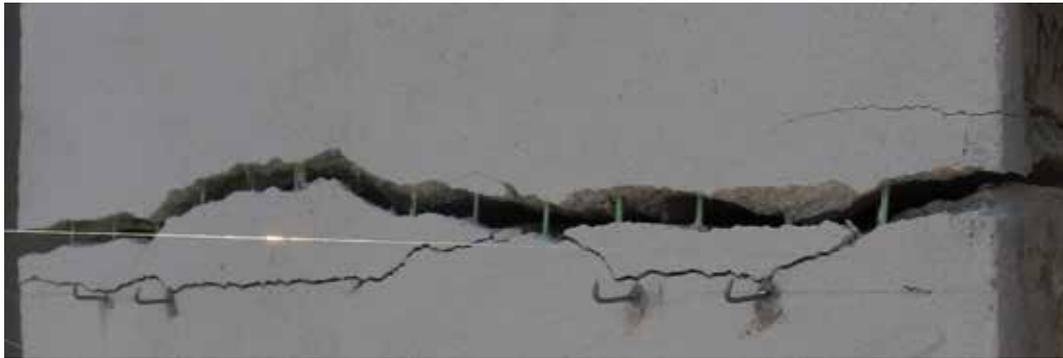

Fig. 07 - *Reinforced sample at the end of the test: detail of the main fissure and the rupture by traction of the net threads*

4.2 EXPERIMENTAL STUDY ON VAULTS IN MASONRY REINFORCED WITH CRM SYSTEM

Description of the project and activities carried out

The experimental study aimed to evaluate the effectiveness of the CRM reinforcement technique on masonry vaults, based on the application, on the extrados or intrados surface, of a reinforced plaster with CRM composite material meshes based on glass or carbon fibers.

The importance of masonry vaults as structural elements, which can have either structural or non-structural functions, is high in the condition of incipient collapse because even minimal movements of these elements can lead to their collapse, with potential consequences for human safety. Based on these considerations and the fact that no experimental campaigns have been carried out regarding the mechanical behavior of these very thin elements, and little research has been done on consolidation systems for these elements, a planning of an experimental campaign was established to investigate the mechanical behavior in the presence of seismic action of vaults and supports, to define the behavior of vaults consolidated with these reinforcement systems, and to define design rules for these interventions.

The activities carried out included the execution of quasi- static cyclic experimental tests on non-load-bearing barrel vaults made of brick and lime mortar, at full scale (radius 2060 mm, width 770 mm). The test setup was designed to reproduce the effects of a cyclic horizontal load proportional to the masses. In particular, eight samples were made: two single-headed vaults (thickness 120 mm) with a ratio of span radius f/r = 0.75 and six thin-shell vaults (thickness 55 mm) with f/r = 0.50. The test samples were prepared at the company's premises. The tests were carried out on-site, in the presence of personnel from the Department of Engineering and Architecture of the University of Trieste. For the evaluation of the material characteristics, some experimental results obtained previously were integrated with those of new experimental tests carried out at the company.

Project objectives

The main objective of the research was to define the role of connections in reinforced vaults to the springers: stainless steel bars embedded in the reinforcement mortar and injected into the lateral shoulders in masonry were used; furthermore, in the case of reinforcement applied to the intrados, cut keys were introduced at the springers, consisting of composite connectors. From the analysis of

the dissipative capacities of the reinforced vault specimens, a significant percentage of dissipated energy emerged, amounting to approximately 61% of the input energy, corresponding to an average hysteretic damping coefficient of 12%. It was found that these values remain approximately constant throughout the cycles throughout the damage spreading phase until collapse.

Main results obtained and subsequent elaborations

The expected results for this campaign were related to obtaining data necessary to describe the behavior of low-thickness and very low-thickness vaulted masonry elements and to define design rules for consolidation interventions with the company's systems, CRM (Composite Reinforced Mortar) - Ri-Struttura System.

Significant results were obtained from a campaign of experimental tests conducted on non-load-bearing masonry vaults at full scale, subjected to quasi-static

cyclic horizontal action. The characteristics of the samples, the materials used, and the experimental test setup were described in detail. All the vaults were made of solid brick masonry (arranged in single header or "in folio" - thin-shell) and natural hydraulic lime mortar, with a span of 3.9 m and a width of 0.77 m. The results of eight tests on masonry vaults were illustrated, including two single-header vaults characterized by a span/radius ratio (f/r) of 0.75 and six thin-shell vaults with f/r = 0.50.

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Fig.08 - *Test setup.*

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For both single-headed and thin-shell ("in folio") types, one vault was reinforced on the extrados, one on the intrados using the CRM technique, and one vault was left unreinforced.

M14 connecting rod to HEB220

UPN 140 for compression

Impost sill

Metal brackets fixed to the floor

UPN 140

Linking bars M14

Main lower beam of the apparatus (heb220)

HEA160 flanged at the ends

Connection bolts M14 at heb220
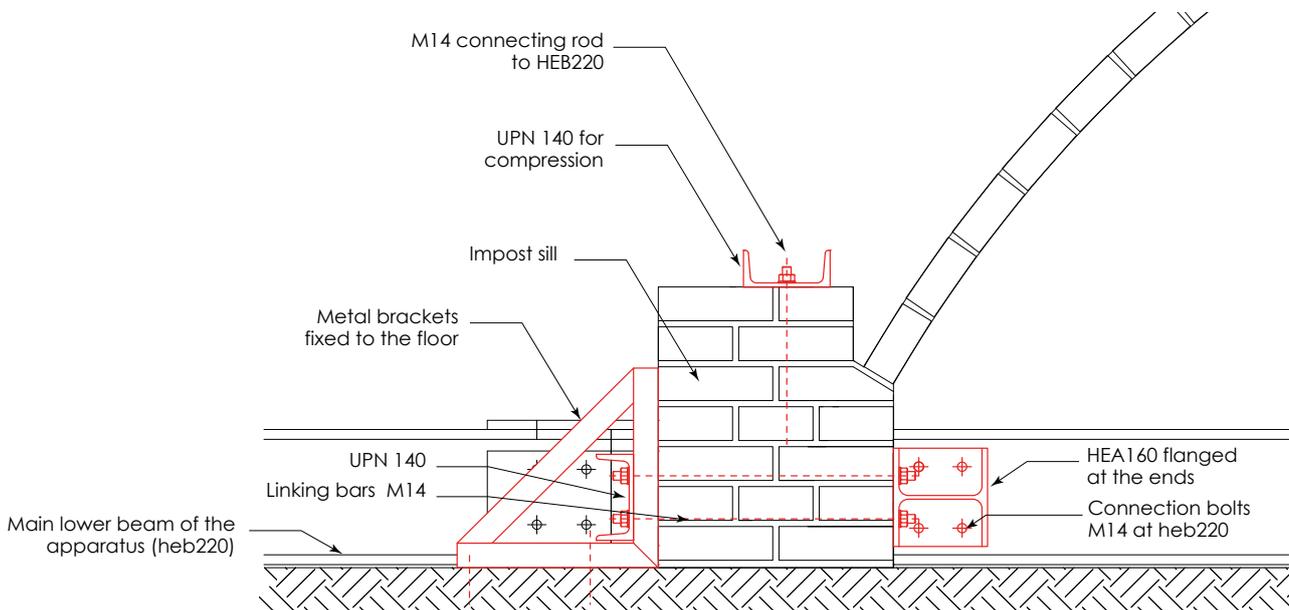
Fig.09 - *Detail of the vault springing*

Additionally, a vault sample was tested where the intrados reinforcement, of the CRM type, was applied over an existing layer of plaster. Special attention was given to the role of connections of reinforced vaults to the springers: stainless steel bars embedded in the reinforcement mortar and injected into the lateral shoulders of the masonry were used. Furthermore, in the case of reinforcement applied to the intrados, cut keys were introduced at the springers, consisting of composite connectors.
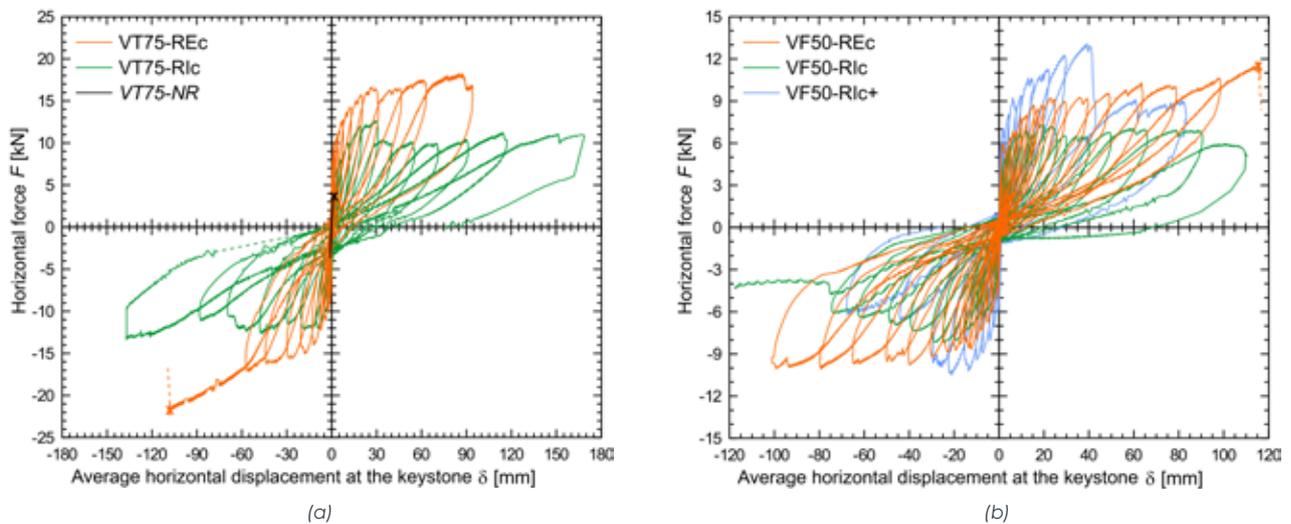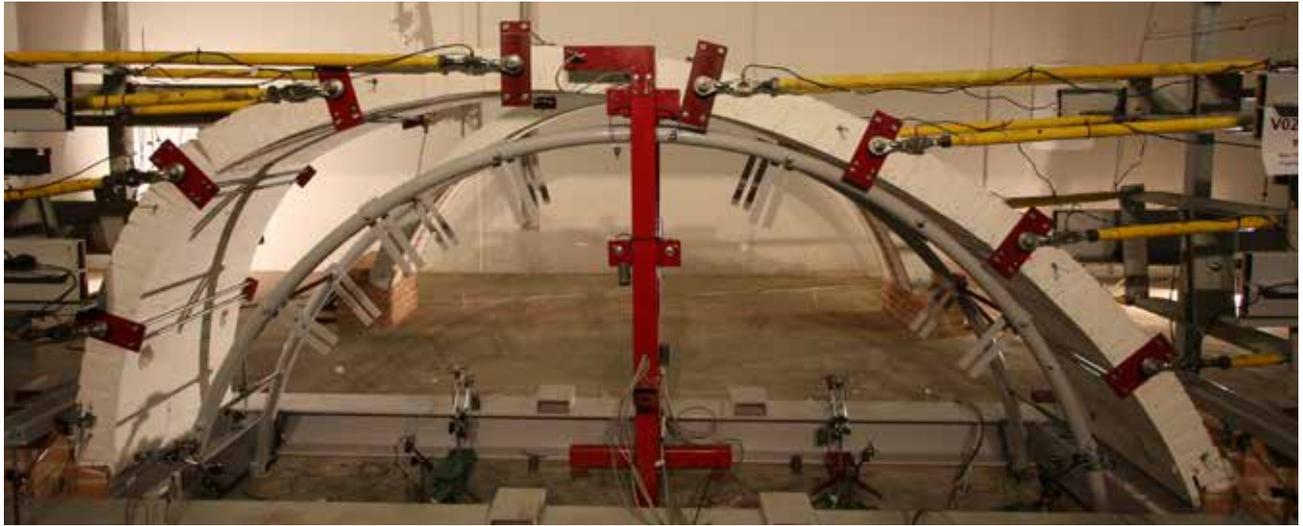
Fig.10 - Confronto tra le curve di capacità dei campioni di volte a tutto sesto (a) e volte ribassate (b).

The obtained results have been very positive, both in terms of increasing the resistance capacity of the consolidated vaults against horizontal actions and in terms of displacement. In the latter case, the increase has even been more than 10 times greater. The implementation of the CRM System, both in intrados configuration and in extrados configuration, has conferred significant tensile strength to the consolidated vault element, resulting in a substantial improvement in mechanical behavior against horizontal actions, also intervening at the constraint level with appropriate structural details. The presence of steel bars has thus allowed the CRM System to be fully effective and to confer resistance capacity and ductility to the tested arches.

THANKS TO

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4.3 THE PROJECTS "SICURA" AND "SISMI"

The set goal of reconciling the preservation of the historical and architectural characteristics of the building with the necessary safety assurance engaged researchers from the Department of Engineering of the University Roma Tre, the Department of Structural and Geotechnical Engineering of Sapienza University of Rome, and the Casaccia Research Center of ENEA, in collaboration with the company, for simulating the mechanisms of disintegration and collapse of historical masonry and the development of new reinforcement technologies. The project was funded by the Civil Protection through the ReLUIS Consortium and by the Lazio Region as part of the SISMI projects (Technologies for improving safety and reconstructing historical centers in seismic risk areas) and SICURA (Sustainable technologies for seismic protection of cultural heritage).

The first research - dynamic investigations on regular masonry

The sample consisted of a wall (equipped with a window), two side walls (one of which had a door near the corner), and a sloping wooden roof. The sample was tested unreinforced and then repaired and reinforced with the CRM System. The L-shaped cross-linked GFRP connectors improved the load transfer capacity from the mesh to the masonry substrate. Introducing the concept of reinforcement optimization, only the most damaged walls were reinforced, and the CRM was applied only on the external face, keeping the internal side unchanged. This solution allows working without evacuating the building or suspending its use (provided that it is sufficient to work only on the perimeter walls). The seismic behavior of the specimen before and after the consolidation intervention is analyzed in terms of progressive damage, failure modes, base acceleration, displacement capacity, and dynamic properties under increasing intensity of seismic input.

Test sample

A full-scale U-shaped tuff masonry structure was tested on the shaking table (Fig. 11-12). It consisted of a front wall 3.30 m wide and two side walls 2.55 m long (right and left). All walls were 3.41 m high with a thickness of 0.25 m and were built with tuff blocks measuring 370 mm × 250 mm × 110 mm, with a compressive strength of approximately 6 N/mm2 and a Young's modulus of 1575 N/mm2 (average laboratory values) and mortar of strength class M2.5 based on prepackaged lime according to EN 1015-11.

Test setup

The tests were conducted at the ENEA Casaccia Research Center in Rome, on a shaking table measuring 4 m × 4 m with 6 degrees of freedom, controlled by four horizontal and four vertical hydraulic actuators.
The sample was secured to the shaking table using ∅20 mm anchoring bars and UPN100 steel beams, positioned through the foundation.
For this investigation, four natural accelerograms were selected, which were recorded by the Italian National Accelerometric Network (**RAN**) during the most intense earthquakes in Italy in recent years and made available for download in the European Strong Motion (**ESM**) database. Each accelerogram is identified by the code of the recording station, namely MRN for Mirandola (2012 Emilia earthquake), **NRC** for Norcia (2016 Amatrice earthquake), **NCR** for Nocera Umbra (1997 Umbria-Marche earthquake), and **AQV** for L'Aquila (2009 L'Aquila earthquake). The input signals were applied in horizontal direction (orthogonal to the front wall) and vertical direction.

Fig.11 - *Seismic reinforcement mode through CRM System applied to two of the prototype's exterior walls.*
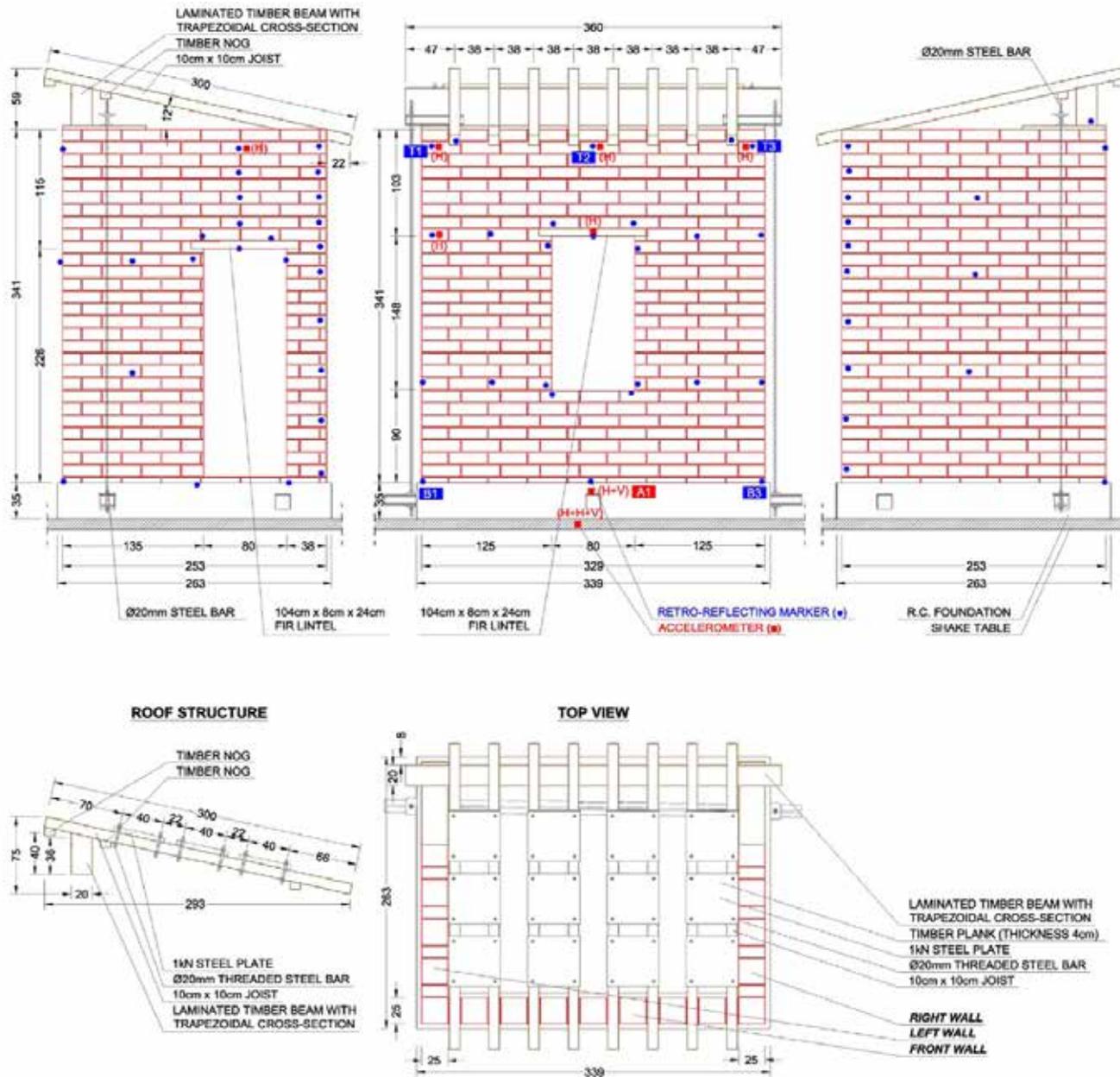
Fig.12 - *Test sample on shaking table and instrumentation used.*

Results of tests on unreinforced and reinforced sample.

The cracking mode exhibited by the unreinforced sample is typical of unreinforced masonry structures (**Fig. 13**) and is related to the arrangement of the masonry components, the presence of openings, and horizontal actions. In particular, seismic loads outside the plane of the main wall and the thrust of the sloping roof at its top were transferred to the side walls due to the good connection provided by the overlap of tuff blocks at the corners (anchorage). The resulting shear loads triggered the development of cracks in the wall weakened by the presence of the door, significantly reducing the development of vertical anchorage. The upper portion of the left wall, prevented from potential diagonal cracking, was involved in the out-of-plane collapse mechanism of the facade. Finally, the local cracking of the upper part of the wall was due to the force transferred by the roof beam and was exacerbated by the lack of vertical compression (essentially unloaded wall).

Fig. 13 - *Crack pattern of the unreinforced sample at the end of the test: overall view (a) and detail of the cracking externally (b) and internally (c) to the left wall*

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The reinforced sample underwent a total of 46 seismic excitation tests and 15 white noise tests. The tests performed in the previous session were repeated to compare the results before and after reinforcement. In this case, no damage developed in the series with **SF** = 0.2 and **SF** = 0.4, while an initial crack was detected on the right wall (the unreinforced one), near the corner with the front wall, after **NCR06** (amax = 0.42g). This crack slightly increased in both amplitude and extent during the subsequent test (AQV06). The first day of testing concluded with **NRC10** (only the first two records were applied with **SF** = 1) and **WHNf**. At the beginning of the second day, the white noise test was repeated (**WHNg**), and the sequence with recordings at natural scale was resumed from the beginning and completed (**MRN10b**, **MRC10b**, **NCR10**, **AQV10**). The crack pattern at the end of this series of tests widened to the right wall and included some diagonal cracks, mainly following the vertical and horizontal mortar joints and crossing the entire thickness of the wall. The cracks were concentrated on the upper part of the wall near the corner with the front wall (**Fig. 13**), due to the loads transferred from the roof, depending on the inertial forces on both the wall and the roof **[13]**.
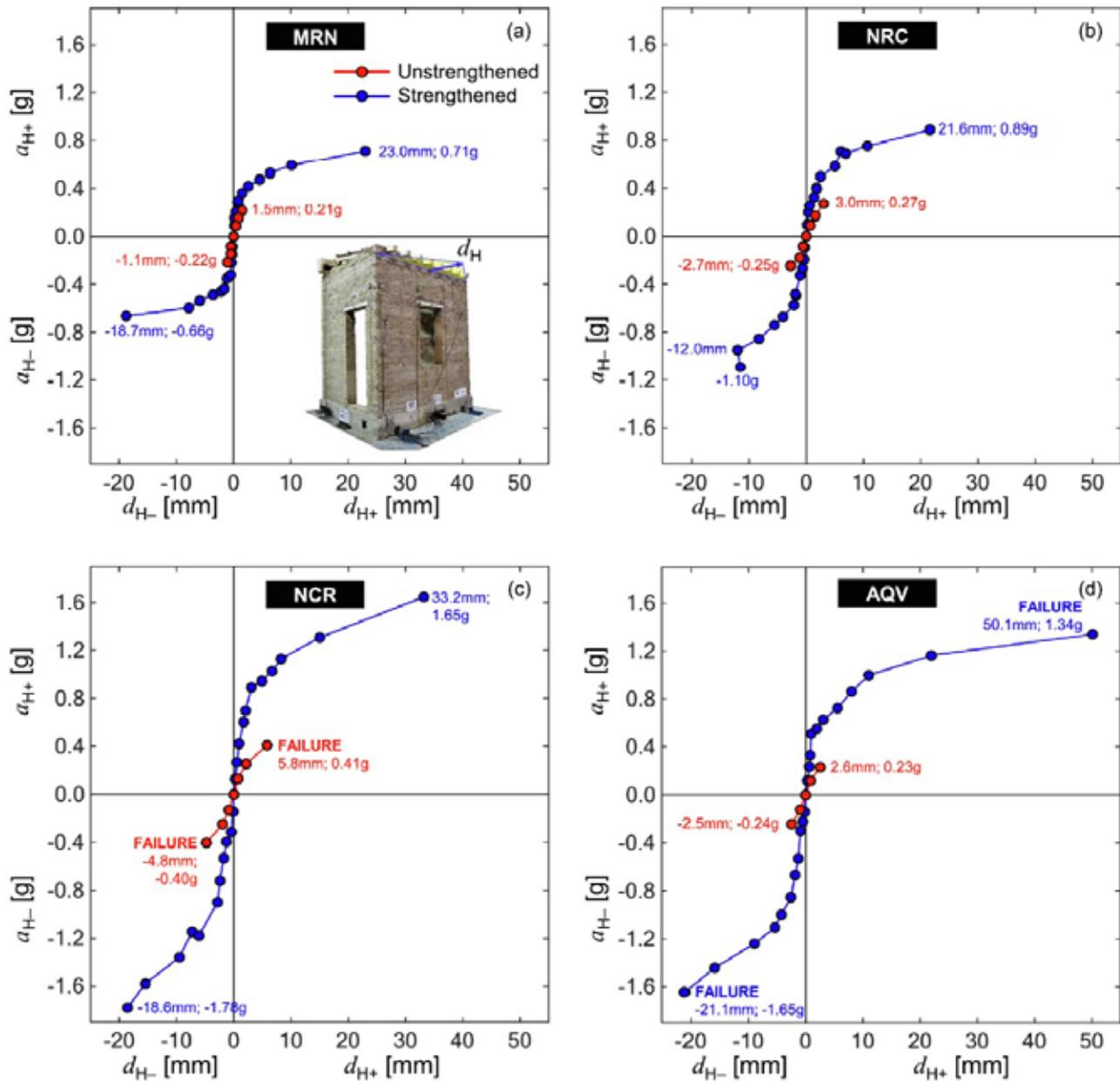
Fig. 14 - *Horizontal component of acceleration vs. out-of-plane displacement at the top under accelerograms according to MRN (a), NRC (b), NCR (c), and AQV (d).*

During the test series with **SF** = 1.2 (amax = 0.90g, **NRC12**), the cracks on the right wall further widened and extended. The consequent increase in the displacement of the upper right corner of the front wall coincided with a significant reduction in the load transfer effectiveness between the two walls, which relied on the arrangement of tuff blocks and the GFRP connectors L = 580 mm inserted during the reinforcement works. Some cracks also developed on the internal side of the facade along the mortar joints (involving the tuff units only locally), near the window and at the base (first bed joint above the foundation), highlighting a possible rotation mechanism (the side wall follows the front wall) and sliding, considering the almost total absence of axial action on that wall. As the unconsolidated wall was about to overturn, a steel tie rod was installed to restore the connection between the upper right corner of the facade and the right wall (**Fig.15c**). The tie rod allowed further tests to be performed with **SF** = 1.4, 1.6, and 1.8. After **AQV18**, the further deterioration of the crack pattern on the right wall and the fall of two tuff blocks from its upper courses required tightening of the tie rod. Furthermore, some cracks appeared on the external side of the front wall, starting from the upper edges of the window and extending towards the upper edges of the wall, due to out-of-plane bending, particularly its upper part (having feared this type of collapse mechanism, the top of the wall had been reinforced with an additional layer of mesh, in this case using a carefully refined corner element to easily accommodate the wooden roof beams).
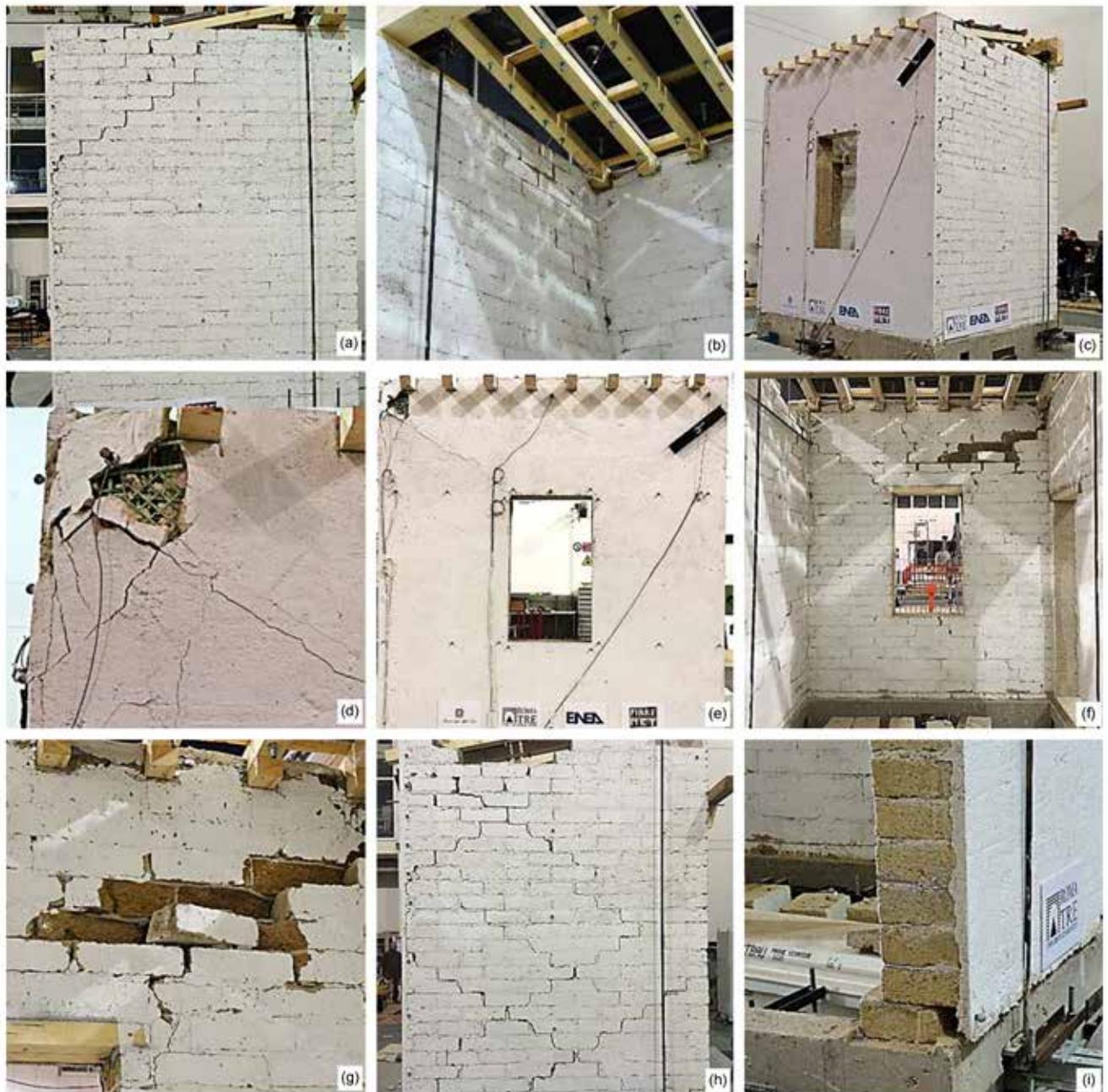
Fig.15 - *Damage in the reinforced sample: internal cracking (a) and external cracking (b) on the right wall AQV120, installation of the steel tie rod after AQV120 (c), detail of the final crack pattern on the left corner of the front facade (d), external cracking (e) and internal cracking (f) of the central wall at the end of the test, detail of the crushing of the internal tuff elements in the wall (g), crack pattern at the end of the test on the right wall (h), sliding of the left wall from the foundation (i).*

The experimental session continued with **SF** = 2.0 and **SF** = 2.2, and the maximum absolute base acceleration reached a_{max} = 1.78g (**NCR22**). Collapse was reached during **AQV22** (amax = 1.65g) and was caused by the failure of the CRM System at the top left corner of the facade (**Fig.15d**).

The GFRP mesh protruded from the substrate (due to excessive deformations imposed by the horizontal thrust of the wooden beams at the top of the front wall), and the corner element opened up, compromising the connection between the left and front walls in that area. Overall, the mesh wires did not break, and no signs of connector breakage or extraction were detected. The mortar of the CRM System was extensively cracked, and some portions were expelled near the top left corner. Additionally, other cracks on the CRM overlay widened (**Fig.15e**). The internal side of the main wall was severely damaged (**Fig.15f**), some tuff units at its top were disintegrated (**Fig.15g**), due to compression stresses occurring when the wall deformed outward. However, the role of widespread integrity provided by the CRM System, albeit only on the exterior of the test building, prevented the collapse of large portions of masonry. Finally, no cracks were detected on the exterior of the left wall, reinforced with CRM, except for a through horizontal crack involving the entire first bed joint (note that no connectors were installed between the walls and the foundation, and this is one of the first turning points in defining the construction details of the CRM System by Fibre Net and the importance they had in the outcomes of the tests on full-scale buildings that will be discussed later). This crack separated the sample from the foundation, allowing sliding, with a residual displacement (out of the plane of the wall) of 115 mm (**Fig.15i**).

The second research seam - dynamic investigations on irregular, poor, and with internal core masonry

The research activities involved experimentation on a shaking table, where walls made with stone elements from the rubble of one of the fractions of Accumoli were reproduced at full scale according to the construction methods observed on-site. The goal was to investigate collapse phenomena such as separation between the facades and disintegration of the masonry induced by an irregular masonry texture and a mortar poor in lime, with very modest mechanical properties (**Fig.16**). The simulation of seismic behavior was carried out by applying to the base of the prototypes the recordings of the main events of the 2016 seismic sequence, in both horizontal and vertical components, progressively scaling the signal amplitude until the collapse of the prototypes.

Fig.16 - *Damage detected in the historic villages of the Central Apennines after the seismic events of 2016-2017.*

The experimentation on the shaking table has already been presented in an article in **[13]**, in which the results of the tests conducted on the unreinforced prototype and the consolidated wall were illustrated.

The article described an innovative seismic improvement technology, resulting from the first test campaign and specifically developed for exposed stone masonry walls. For these types of masonry, it is not always sufficient to intervene with devices to restrain local mechanisms such as chains or top reinforcement beams in reinforced masonry; most of the time, it is necessary to provide intervention distributed on the wall surface to counteract the disintegration of the wall with separation between the two facades, without being able to use reinforced plasters, which would compromise the exposed face of the masonry.

The reinforcement technology developed and tested in the laboratory consists of applying preformed polymer connectors reinforced with carbon fiber (CFRP) transversely into the stone elements of the external facade (exposed face), without penetrating the entire thickness of the wall so as not to affect the internal surface. The use of 6 mm diameter connectors ensures that, after grouting the holes, the reinforcement remains hidden inside the wall, preserving the exposed face characteristics of the masonry. The application from the external facade leaves the interior of the building undisturbed, without compromising any frescoes or decorations and without interrupting the functionality and use by occupants.
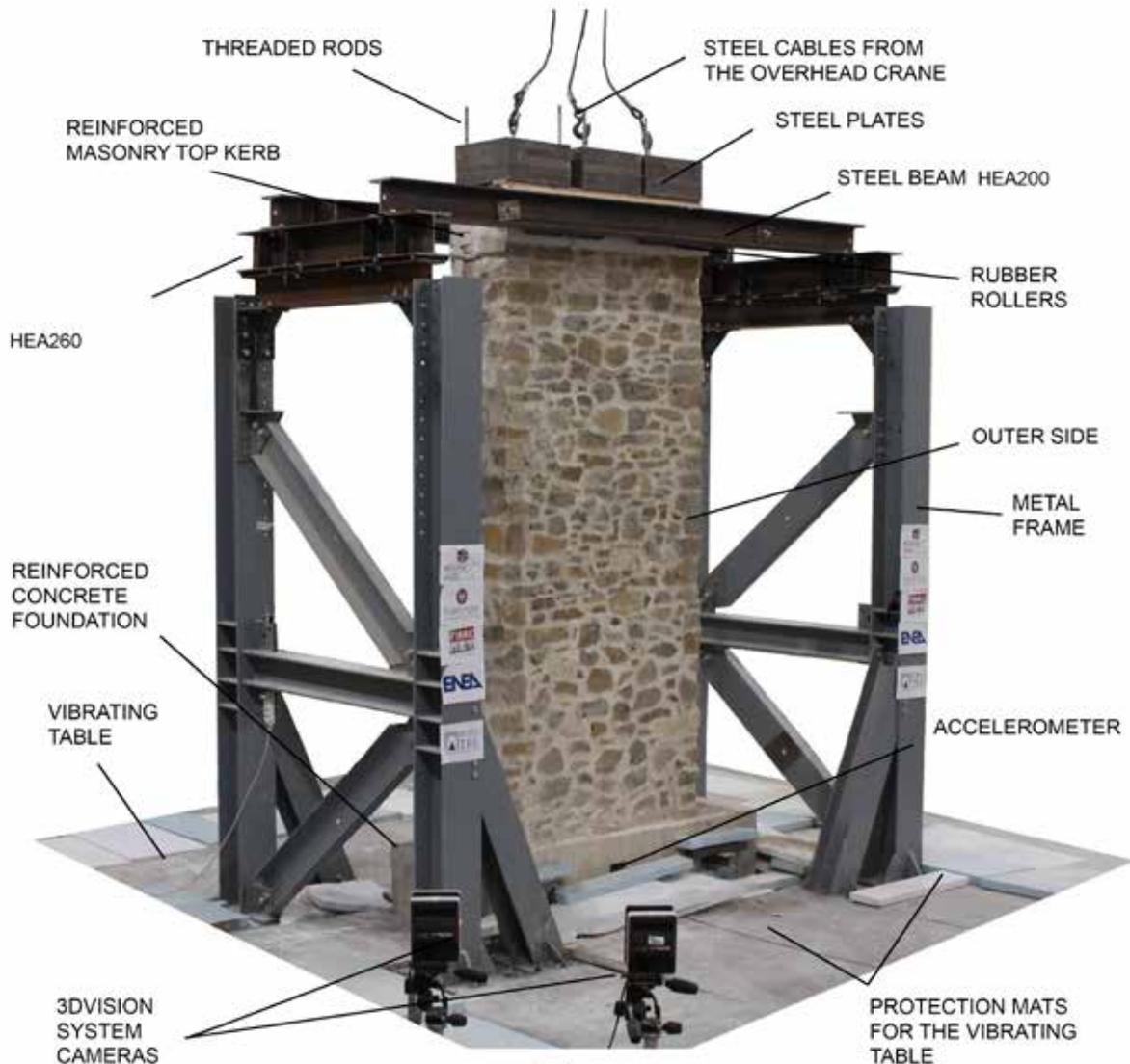

Fig.17 - *Experimental setup.*

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Test sample

The construction methods of the prototypes followed local practice, with two separate facades, a central core composed of smaller and hollow stone elements, in the absence of through stones. Overall, the study involved the creation of two prototypes, having the same geometric and mechanical characteristics: thickness 0.5m, width 1.6m, and height 3.7m, to achieve a height/thickness ratio of about 7.5, similar to what was found on site. In order to replicate the architectural features of the buildings, the front side of the walls was built with exposed stone masonry, while the back side was finished with a regular lime mortar plaster. A first sample (referred to as UR) was tested unreinforced, while a second sample (referred to as CC) was reinforced with the proposed CFRP transverse connectors.

Test setup

The test setup was designed to subject the prototypes to out-of-plane vertical bending and consists of two steel braced frames, rigidly bolted to the shaking table, supporting two beams equipped with hard rubber rollers, placed in contact with the front and back faces of the prototype, in order to provide lateral restraint at the top of the wall (**Fig.17**). The rollers allow free vertical displacement and rotation of the wall while providing lateral restraint, albeit with a certain flexibility similar to that offered by a roof slab, at the top of the wall in the out-of-plane horizontal direction.

Design of the sample consolidation

The seismic improvement solution involves the preliminary regeneration of the stone masonry through consolidation injections with hydraulic lime mortar, aiming to make the masonry pier as monolithic as possible. Subsequently, carbon fiber reinforced polymer (CFRP) connectors are installed by drilling into the stones of the exposed facade and made integral not with resin but with special mortars with high workability. The choice of carbon fiber has allowed for a reduction in connector size (and therefore hole size) thanks to its high mechanical properties (strength and elastic modulus), minimizing the impact on the wall and ensuring durability in the alkaline environment induced by lime-based mortars. It is estimated that at least 50% of the installed connectors, after passing through the internal core of the wall, have reached the stone elements of the internal facade (whose arrangement was hidden by plaster), thus achieving an effective transverse connection while leaving the internal side of the wall undisturbed. Therefore, in principle, this reinforcement technology also does not require evacuation of the building during seismic improvement works. The holes on the exposed facade side are finally sealed with injection mortar to conceal the connector heads.

Results of the tests on the shaking table

The application of seismic inputs with progressively increasing scaling factor, as planned in the experimental protocol, allowed observing the evolution of damage until collapse. The unreinforced prototype (**UR**) exhibited significant cracking already in the test sequence with **SF**=0.6 (base acceleration 0.22g), associated with a clear separation between the two facades, and collapsed during simulation **CNE08** (seismic input **CNE** with **SF**=0.8) due to disintegration of the exposed facade (unplastered, **Fig.18**). The maximum acceleration recorded at the base of the wall was 0.45g. On the other hand, the wall reinforced with preformed CFRP (Carbon Fiber Reinforced Polymer) connectors remained essentially elastic and damage-free until the test cycle with **SF**=0.8. The first hairline cracks appeared after **AMT10**, but significant cracks were only detected in the test sequence with **SF**=1.2 (base acceleration 0.59g). The experimental results thus demonstrate the effectiveness of the proposed reinforcement solution, which would have allowed the stone wall to withstand substantially without damage all three seismic events whose recordings were used for shaking table simulations. With further increase in seismic action, the first collapse occurred in simulation **NRC14**, with detachment of a stone element below the top beam. In the subsequent simulation (AMT14), additional stones collapsed, both from the external and internal sides, effectively separating the wall from the upper coping (**Fig.19**).

Fig.18 - Progressive damage of the UR sample due to separation between the facades and disintegration of the external exposed stone masonry facade.

The peak acceleration value recorded at the base of the wall during simulation **AMT14** was 1.03g. Therefore, the seismic improvement compared to the unreinforced wall (**UR**) in terms of acceleration increase at the base is 127%.

Fig.19 - Progressive damage of the CC sample due to displacement of stone elements below the top beam, without separation between the facades.
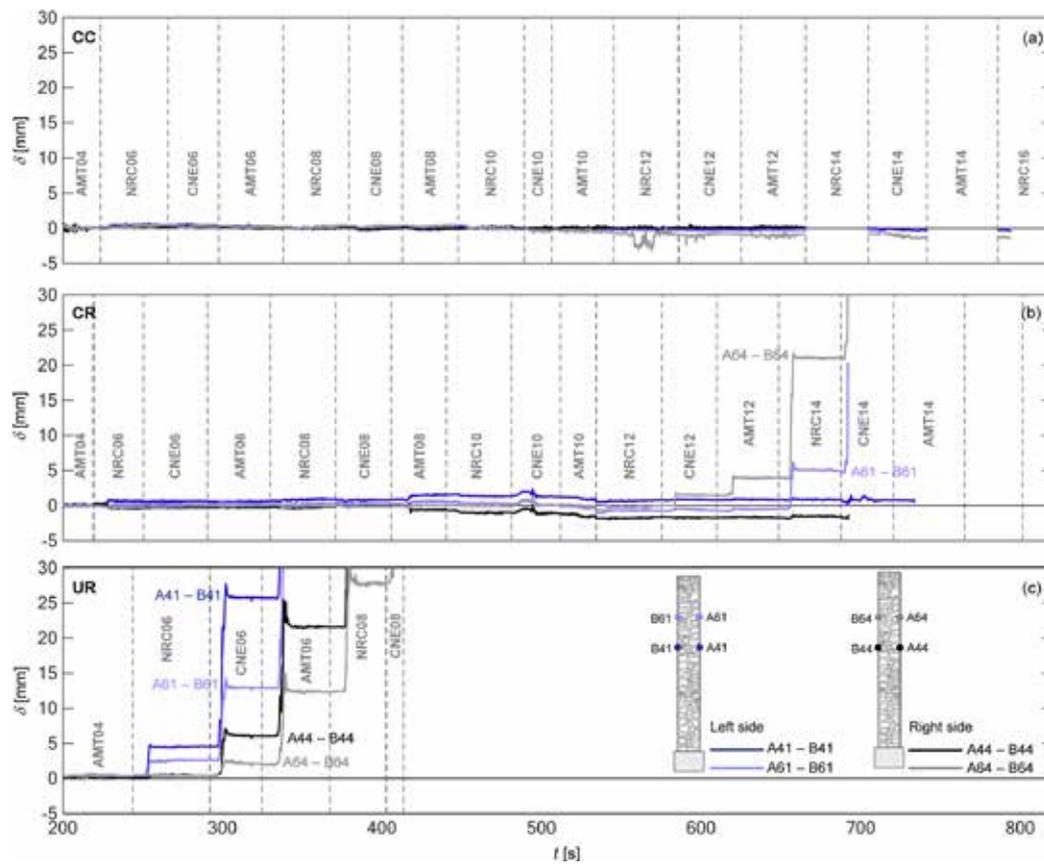
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Fig.20 - *Relative displacements between the two facades in the sample consolidated with carbon bars and injections (CC), consolidated with the Reticola Plus System (CR), and unreinforced control sample (UR).*

THANKS TO

4.4   THE PUSH 'O VER PROJECT

The research path undertaken, aimed at defining the analytical formulations governing the behavior of masonry elements consolidated with the CRM System and giving substance to the concept of optimization mentioned in previous pages, has received significant validation through the "PUSH 'O VER" Project, in collaboration with the University of Camerino - Prof. Dall'Asta, ENEA - Prof. Clemente, University of Rome La Sapienza - Profs. Liberatore and Sorrentino, DOING Ingegneria, EAS Ingegneria, Di Emidio Progetti, CMP, Labortec Ingest, and the construction company Gaspari Gabriele.

In this comprehensive research project, the main objective was to evaluate the resistance capacity, against seismic actions, of an existing full-scale masonry building, also drawing inspiration from tests with comparable-sized masonry samples [15], [16], [17]. This was achieved by applying a quasi-static horizontal thrust to two identical portions of the building, one restored and the other subject to structural reinforcement with the CRM System, using a metal carpentry structure as restraint on deep foundations, both purpose-built for the research project. To highlight the difference in behavior and response of the two building bodies, the CRM System,

applied on both faces of load-bearing walls at the first level, was applied only on the external face at the second level: while the first solution ensures a more performant result, the second, also in view of seismic improvement interventions "superbonus 110%", allows work on at-risk buildings without requiring the transfer of people and functions present therein.

The experimental tests allowed us to study the response in terms of elasto-plastic behavior until the collapse of the two-story building, evaluating the increase in mechanical performance of the consolidated configuration compared to the unreinforced one. The company's contribution was not limited to providing its technologies and laboratory equipment, allowing the use of measurement instruments for assessing the evolution of structural elements' damage (complex positioning of instrumentation on test samples), but also manifested in active technical-scientific collaboration with the University of Camerino and ENEA. The company's technical team provided support in both the design phase of the consolidation system and test setup and in carrying out experimental activities, monitoring system, and interpreting experimental data.

Test sample

The original building, used to derive the two investigated structures [18], is located in an area affected by the earthquakes in Central Italy in 2016. It was built in the first half of the nineteenth century and was in poor condition, with widespread and branched cracks due to the aforementioned seismic events. The building is a two-story clay-brick masonry structure, with an almost square shape of 11.42 m x 10.87 m, and an interfloor height of 2.80 m (Ground Floor) and 3.00 m (First Floor).

Figure 21 shows the ground floor and first floor plans of the original structure and a photo of the main facade of the building before the interventions and tests. The masonry structure is made of bricks with 2 headers connected by a poor mortar with an overall thickness of 0.25 m. The left part of the first floor of the building consists of a jack arch supported by I-beams, while the right side is made of joists with prefabricated reinforced concrete "I" beams. The roof is constructed with wooden elements, while the shallow foundation is in masonry. The plan distributions of the walls identify two seismic masonry cells separated by a staircase connecting the ground floor with the first floor. Once the central staircase and the rear outbuilding were demolished, this structural configuration allowed the building to be divided into two similar portions, called Building 1 and Building 2, characterized by the same dimensions and resistance to horizontal forces (**Fig.21**). The seismic damage suffered by Building 1 was repaired by applying standard intervention techniques (i.e., cracks were repaired with patching technique), while Building 2 was consolidated by applying the CRM System of Fibre Net, using mesh, corner elements, connectors, thermosetting resins embedded in lime and cement-based mortar. The first floor of both buildings is consolidated with a reinforced concrete slab connected to the old floor and the existing perimeter walls; this allowed the horizontal forces to be distributed evenly among the existing vertical walls. The wooden element roofs are replaced by internal planar truss steel structures, connected to the external masonry walls through reinforced concrete ring beams that act as rigid diaphragms. To restore the load applied from the roof to the masonry walls, five water-filled tanks are used, arranged on the top bracing structure of the building. Possible out-of-plane mechanisms of the walls are blocked by introducing tie rods anchored with external steel plates at intermediate and upper levels.
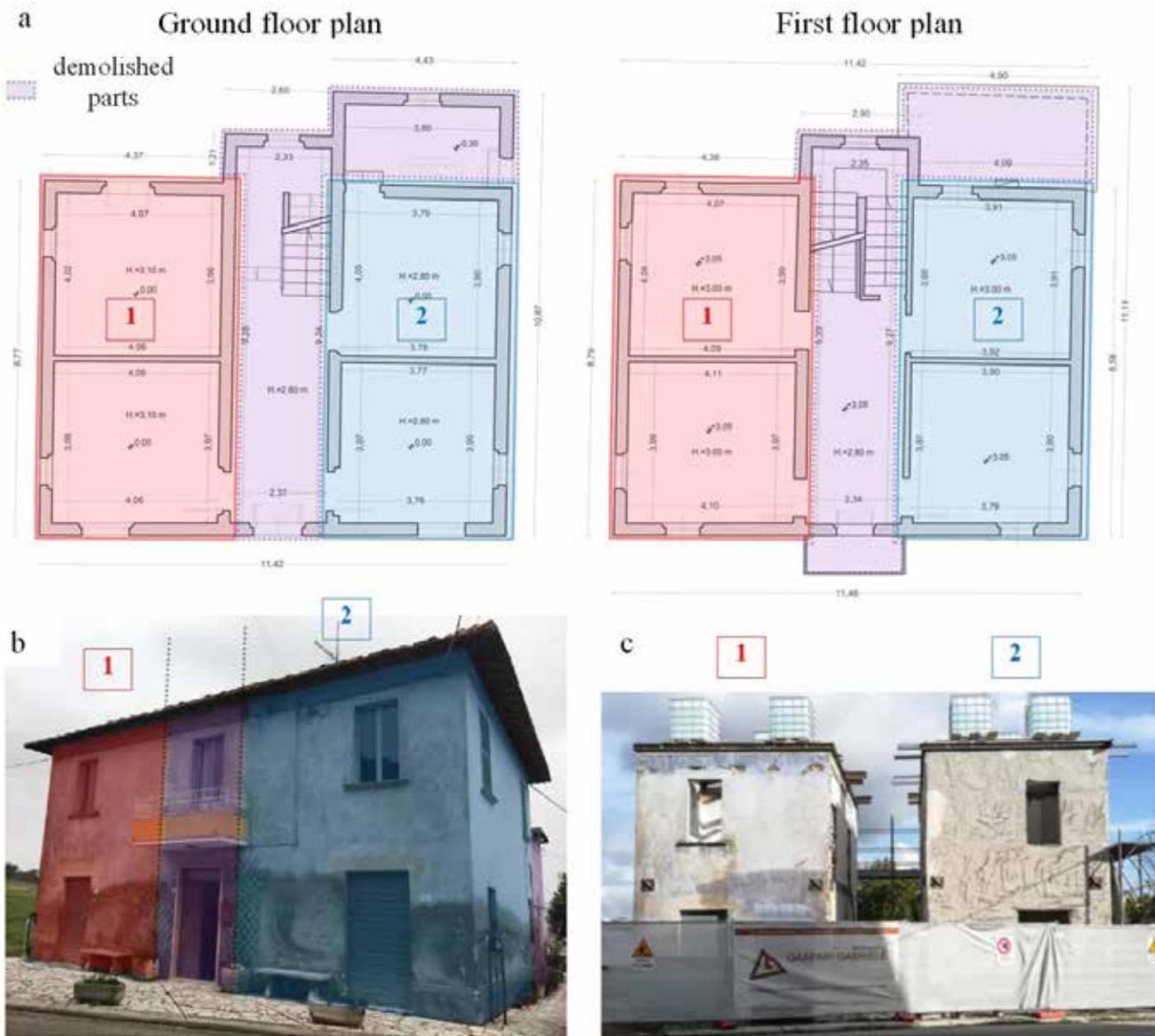
Fig.21 - (a) Ground Floor and First Floor of the building; (b) original building; (c) buildings created for the execution of the tests.

Test setup - conception and structural design

The in-situ tests required a sturdy bracing frame for applying horizontal loads using hydraulic actuators. For this purpose, a braced steel frame was constructed behind the test buildings. The pushing device consisted of two parts: a steel bracing frame and a triangular frame with a sliding prism, as shown in **Fig.22**. To apply the pushing force, two hydraulic jacks with a capacity of 5000 kN each and equipped with load cells were used. A new foundation was specifically built for the tests, consisting of a reinforced concrete raft supported by 8 foundation piles with a diameter of 0.8 m. The pushing device was designed to allow for assembly/ disassembly and movement in case it was decided to subject other buildings to horizontal pushing tests. The ratio between the positions of the contact points on the building and the height of the hydraulic jacks defines the distribution of the applied forces. Here, the ratio between upper and lower forces is 1.28, to reproduce an inverted triangular pushing profile **[19]**.
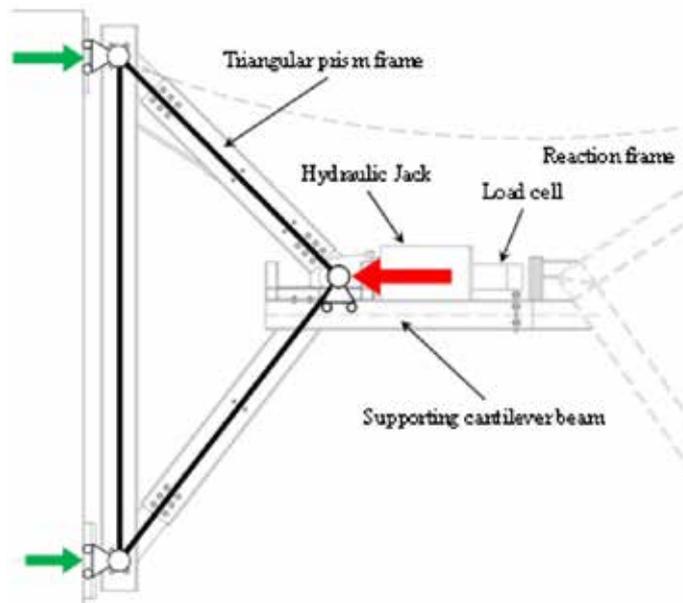
Fig.22 *- (a) Test apparatus; (b) Assembly of the pushing triangle;*
(c) Static scheme of the pushing triangle

Experimental test results

Both structures, namely the original building (Building 1) and the consolidated one (Building 2), were subjected to the action of incremental lateral forces applied to the intermediate and upper floors using the pushing device. These forces were applied in an inverse triangular configuration, representative of a seismic force distribution that is proportional to a linear-shaped pushing profile. Hence, it is possible to highlight the stiffness variations of both buildings during the tests. The secondary loading phases were initially determined in terms of force increments, **ΔF**, which better describe the elastic behavior of the buildings. The **ΔF** used were lower for Building 1 (almost 30 kN each) and higher for Building 2, almost 100 kN. Once the buildings exhibited a transition from elastic to inelastic behavior, i.e., when the force became nearly constant while displacements continued to increase, the steps were updated through displacement increments, **Δu**, of 5 mm for each step. The results, presented here, are expressed in terms of force envelopes. The force input is the sum of the forces applied at both levels of the buildings, while the displacement is the average recorded by the displacement transducers installed at the roof level of the buildings.

Results obtained on the non-consolidated building - Building 1

Figure 23 shows the cracking pattern characterizing Building 1 and Building 2 at the end of the horizontal push tests. The maximum load level reached by Building 1 is 396 kN, while the maximum displacement is 53.5 mm. The cracks, highlighted by continuous red lines, are located both near the upper reinforced concrete distribution beam and the openings, with a sub-diagonal and vertical development. These cracks indicate the portion of the masonry subjected to compressive forces (crack development due to tensile forces induced by the formation of compressed masonry), which is mainly the central masonry portion located between the openings, as well as sliding due to the shear induced by the rigid upper concrete beam. Focusing on the capacity curve, an initially rather stiff elastic behavior can be observed, up to a lateral force of 335 kN with an displacement of nearly 5 mm, while the post-elastic behavior is characterized by a displacement of 47 mm. The maximum drift recorded during the test is close to 1.1% of the height of the upper floor. It is worth noting that **[20]** associates the final displacement for a seismic action with a probability of exceeding 5% in 50 years, with a chord rotation ranging from 0.5% to 1.0% for masonry subjected, respectively, to shear/sliding and flexural compression in the plane. The drift obtained in Building 1, together with the defined cracking pattern, emphasizes the satisfaction of the building's performance when subjected to lateral loads towards the Ultimate Limit State (**ULS**). The tests were interrupted once a significant reduction in the building's capacity was recorded, avoiding significant detachment of masonry or parts of it.

The results obtained on the consolidated building - Building 2

Figure 23b refers to the building consolidated using the CRM Fiber Net System. It can be observed that for this building, the openings were farther away from the pushing device compared to Building 1. However, even in this case, the cracks highlighted in **Figure 23b** emphasize the portion of the masonry subjected to compression, while no sliding due to shear appears at the floor levels (intermediate and roof), also due to the presence of an angle positioned as on the top of the tuff building subjected to shake table testing.

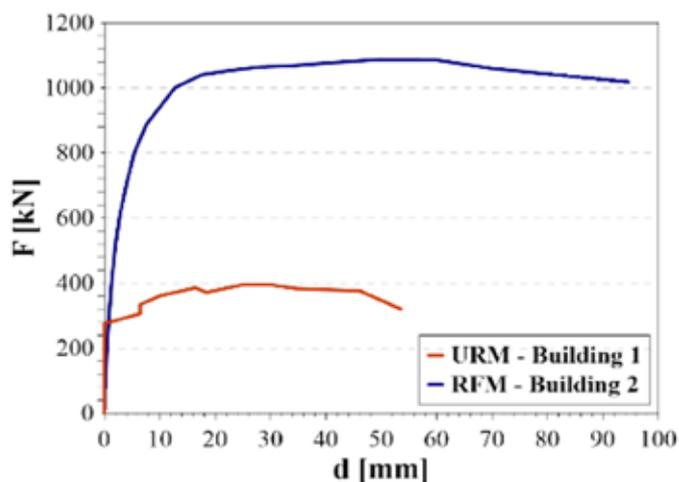*Fig.23 - Building 1 (a) and Building 2 (b): comparison between crack patterns at the end of the test.*

Fig.24 - Curva Push 'O Ver of the non-consolidated building (1) and the consolidated building (2).

Furthermore, the cracks are considerably thinner and less widespread on the reference facades compared to those of Building 1. Regarding the capacity curve, it is observed that the maximum force reached by the building is 1087 kN, which is 2.74 times that reached by Building 1. In terms of displacements, the maximum recorded in Building 2 is 1.8 times higher than that of Building 1, namely 94.6 mm. Regarding the transition from elastic to inelastic behavior, in Building 2 it occurs at a load close to 1000 kN and a displacement of 12.7 mm. This load is almost 3.0 times the elastic load of Building

1. The interstory drift is 1.8% at the First Floor and 1.3% at the Second Floor. Furthermore, the ultimate capacity seems to be associated with a ductile mechanism (flexure) rather than shear sliding or diagonal cracking, which are brittle failure mechanisms, especially for non-consolidated masonry **[21]**. **Figure 25** shows some details regarding the crack pattern of the rear wall of Building 2 and the uplift that the building underwent towards the end of the push tests, considering that it was not possible to effectively intervene in the foundation - through rigid concrete corbelling and adequate connecting bars between this and the CRM reinforcement - due to time constraints: after the consolidation works, indeed, the dynamic behavior of the building when subjected to lateral forces was more influenced by a rigid rotational-translational motion, rather than pure shear. The uplift of the building occurred near the push device, where the building was subjected to tension/flexure out of plane, while the wall opposite the device was subjected to compression/crushing. The test was interrupted when the uplift was almost 100 mm and the phenomenon had involved 2/3 of the shear walls (as per setup) of the building. It is emphasized how buildings of this type are often characterized by shallow foundations, on which it is practically impossible to determine and verify the real effectiveness of the connection: consequently, even in this case, the fundamental role of construction details, which allow the applied system to operate even more effectively, can be glimpsed.

Fig.25 - Building 2 Consolidated: (a) cracking due to torsional effect from modified setup for building uplift; (b) lifting of the front corner of the building.

As a completion of the experimental tests, detailed numerical simulations were conducted using concentrated plasticity springs, calibrating the strength and drift values available in the literature **[22]**.
The intervention solution exclusively on the external face of the masonry, to ensure the internal functionality of the building, had already been tested during a shake table test at the SITEC laboratory of the ENEA Casaccia Research Center as part of a research conducted by the University of Roma Tre (contribution by Prof. G. De Felice and PhD S. De Santis previously mentioned) and finds its completion in the European Interreg Project 2020-2022, Italy-Slovenia cooperation called CONSTRAIN - Sharing and application of innovative strategies for seismic protection of masonry buildings, which Fibre Net, as a Project Partner, won along with the University of Trieste (Lead Partner), the Venetian company Restauri Costruzioni S.r.l., the University of Ljubljana, the construction company Kolektor D.o.o. and Igmat d.d, Inštitut za gradbene materiale.

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4.5   THE CONSTRAIN PROJECT - INTERREG 2020-2022

The CONSTRAIN project has developed new methods of consolidation for existing masonry buildings that can be applied by intervening only from the exterior of the buildings themselves. The proposed reinforcement technique is based on the application from one side of the masonry of reinforced mortars (CRM System - Composite Reinforced Mortar, RI-STRUTTURA by Fibre Net) and innovative anchors (artificial "diatoni", i.e. transvesal elements connecting both facades of a wall). To verify the effectiveness of the proposed technique, an extensive experimental campaign was conducted, which involved cyclic tests on such reinforced masonry elements, including shear-compression tests (masonry piers), flexural tests (spandrels), and out-of-plane bending tests (walls).

Three types of masonry were considered in the tests: rubble stone with two faces and single and double-leaf brickwork. Finally, a test was carried out on a full-scale, two- story above-ground stone masonry building to verify the effectiveness of the technique under real conditions. The results of the tests clearly show that the proposed strengthening significantly improves seismic performance. Numerical models were then calibrated with the experimental results, and numerical simulations were performed on the behavior of a significant masonry building to highlight the level of seismic performance that would be achieved with reinforcement applied only from the exterior of the perimeter walls or on both sides of all load-bearing walls.

Project objectives

The project is based on the synergy of expertise in the productive, executive, and research fields, to promote innovation in the field of structural consolidation interventions and disseminate the knowledge and experiences acquired to increase the know-how and competitiveness of operators in the building sector. Most of the techniques proposed in the past for seismic retrofitting of masonry buildings are generally invasive, involving the evacuation of the building during the consolidation intervention.

In this sense, the project has developed a highly effective intervention technique with low impact on residents and activities taking place in the buildings, foreseeing intervention from the outside with mortar reinforced with a mesh of composite material GFRP adequately connected, in the case of multi-leaf masonry, with appropriately anchored artificial "diatoni" to the reinforcement, to avoid separation of the leaves/ disintegration of the wall.

Experimental tests

An extensive experimental campaign was conducted to understand the effectiveness of the proposed technique **[23]**. The behavior of the reinforced mortar was analyzed in a previous study **[21]**, **[24]**, **[25]**, while in this chapter, the construction details for an effective application of the CRM System from one side are addressed. The campaign includes tests on individual full-scale masonry elements, subjected to both in-plane and out-of-plane loading, and a

quasi-static cyclic test on a two-story stone masonry building. All tests performed were of the quasi-static cyclic type, allowing for the demonstration of energy dissipation capacity at each cycle. In addition to direct measurements of various relative displacements, an optical displacement reading system was used on the surface of the wall, enabling the detection of crack formation and extension in the masonry.

Shear-compression tests on masonry piers

The shear-compression tests simulate the stress state of a masonry pier (**Fig.26a**). The results plotted in **Fig.26b** concern samples of double-leaf stone masonry in unreinforced configuration (black curve), reinforced from one side (orange curve), and reinforced from both sides (blue curve); they clearly highlight the higher strength (60%-120%) and displacement capacity (150%-400%) of the reinforced masonry, respectively for reinforcement from one side and from both sides **[26]**.
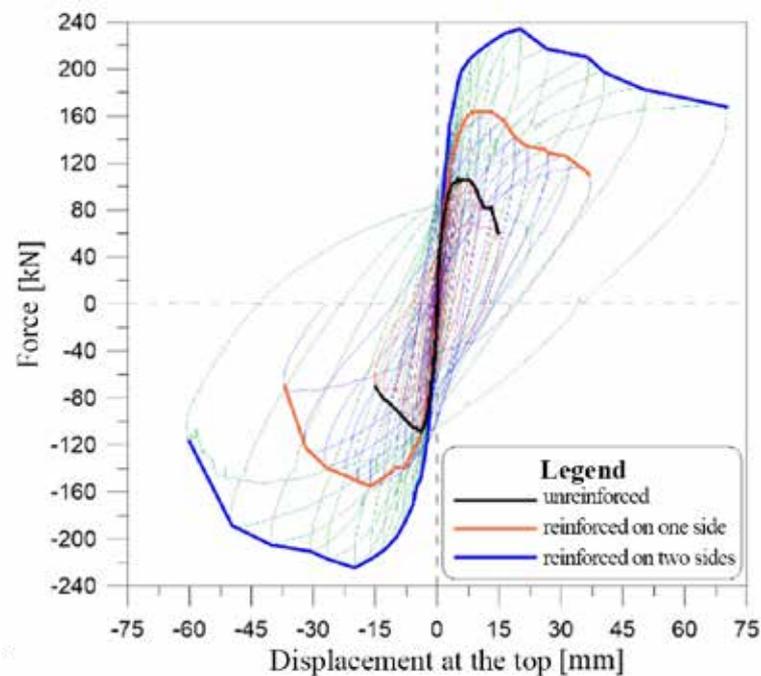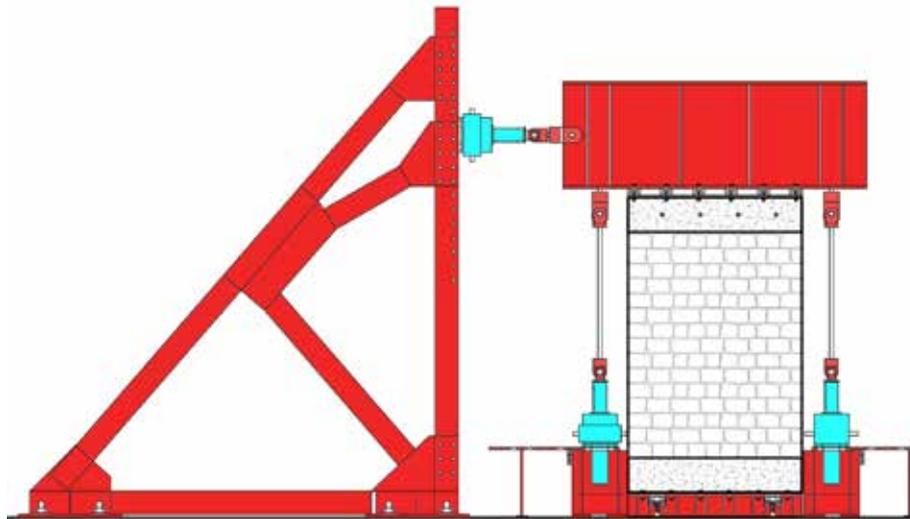
Fig.26 - *Shear-compression test (a); Comparison between unreinforced sample and reinforced samples (b), for a double-leaf stone masonry.*

Tests on shear-bending of lintels above windows/spandrels

Tests on shear-bending simulate the stress state in the lintels above the window openings of a masonry wall (**Fig. 27a**). The results depicted in **Fig. 27b** concern solid brick samples with two headers in unreinforced configuration (red curve) and reinforced on one side (black curve). A significant increase in strength (over 30%) and a substantial enhancement of capacity in terms of displacement (almost tenfold) are highlighted in the reinforced masonry.
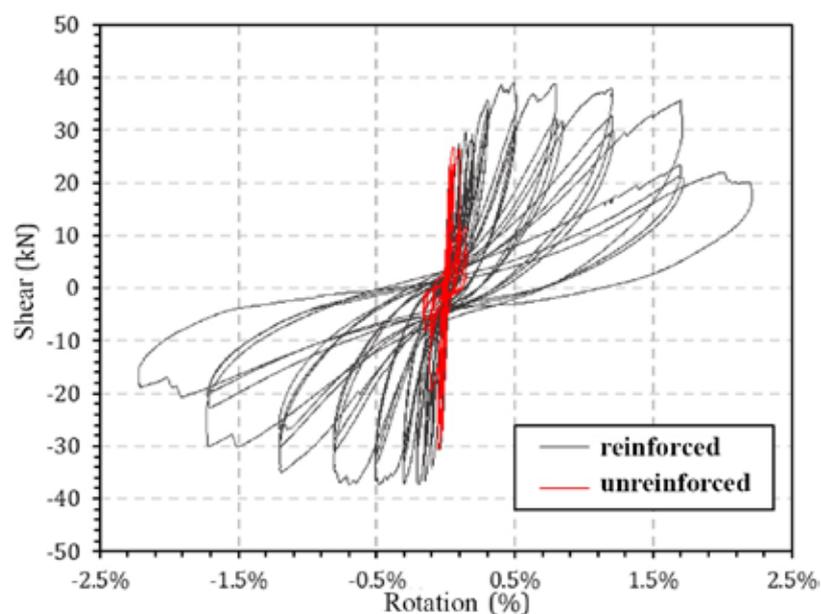
Fig.27 - Shear-bending test photo on lintel above window (a); Comparison between unreinforced sample and reinforced sample (b), for solid brick masonry with two headers in single leaf configuration.

Tests on a pilot masonry building in rubble stone

The tests on the pilot building in **Fig.28a** have shown a significant increase in performance between the unreinforced and reinforced cases. The strength has increased by over 100%, and the displacement capacity has nearly quadrupled **[28]**; even the dissipative capacity per cycle is significantly higher in the reinforced building, as clearly indicated by the diagram in **Fig.28b**.
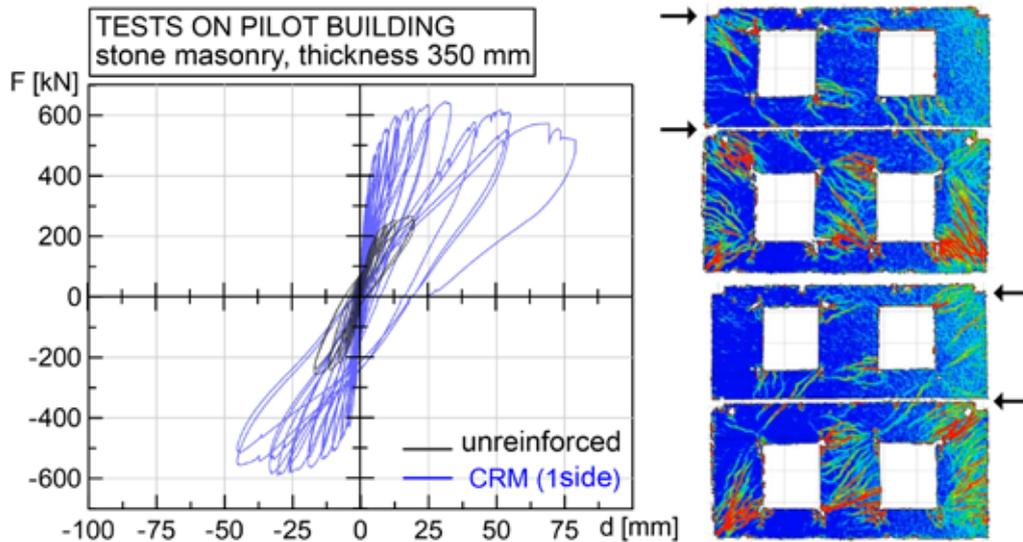
Fig.28 - *Pilot building undergoing experimental testing: view (a), load-displacement diagram at the top, and crack pattern of the reinforced sample at the end of the test (b).*

Scan or click the **Q R C O D E** to watch the video

THANKS TO

4.6 NUMERICAL DETAILED MODELING IN THE DEFINITION OF DESIGN ANALYTICAL FORMULATIONS

The great variability in the combination of materials, geometry, loading patterns, and boundary conditions makes it practically impossible to experimentally cover all possible configurations. Therefore, a numerical approach is essential as it allows for investigation into a wider range of more complex configurations and also enables optimization of the reinforcement intervention.

Given these premises, an extensive numerical study on the structural performance of masonry reinforced with CRM has been developed, focusing

on nonlinear static analysis. Adopting a multi-level approach, various modeling strategies have been calibrated, varying the scale of investigation **(Fig. 29)**: starting from the detailed-level model (**DLM**) for small-scale tests, followed by an optimization procedure to obtain a more computationally efficient multi-layer model (**MLM**) based on layered elements for intermediate-scale tests, and finally, the calibration of a simplified lumped plasticity model (**LPM**) for global analysis.
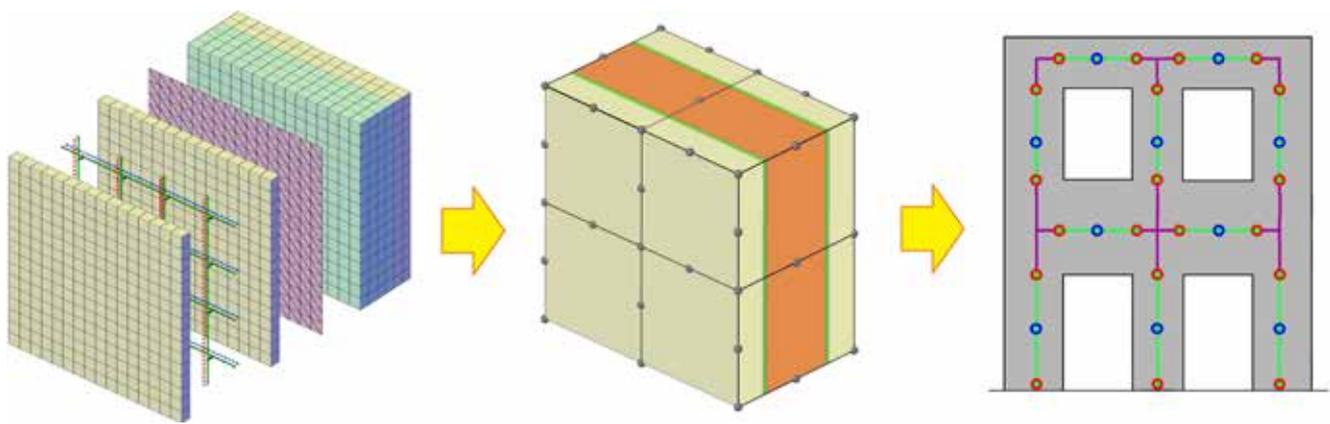
Fig.29 - The multi-level approach adopted for the numerical modeling of CRM reinforced masonry: (a) detailed level model DLM, (b) multi-layered intermediate model MLM, and (c) simplified lumped plasticity model LPM.

The detailed level modeling (DLM) (**Fig.29a**) is characterized by a very dense mesh (~15 mm), consisting of 8-node solid elements used to represent masonry and plaster, and one-dimensional truss elements for the reinforcement wires. Point-to-point interfaces connect orthogonal wires at intersections of the mesh; line interfaces connect reinforcement wires to mortar element edges; surface interfaces couple mortar element faces with masonry faces. The model accounts for the nonlinearities of individual materials (such as wire tension failure, cracking, and

crushing of plaster and masonry), as well as their interactions (such as wire node failure, detachment of wires from mortar, and mortar from masonry support). The DLM model was calibrated based on experimental tests on individual components and interfaces (**Fig.29a**) and validated by comparison with tests on CRM specimens (**Fig.29b**) **[4]** and elementary masonry samples (**Fig.29c**) **[5].**
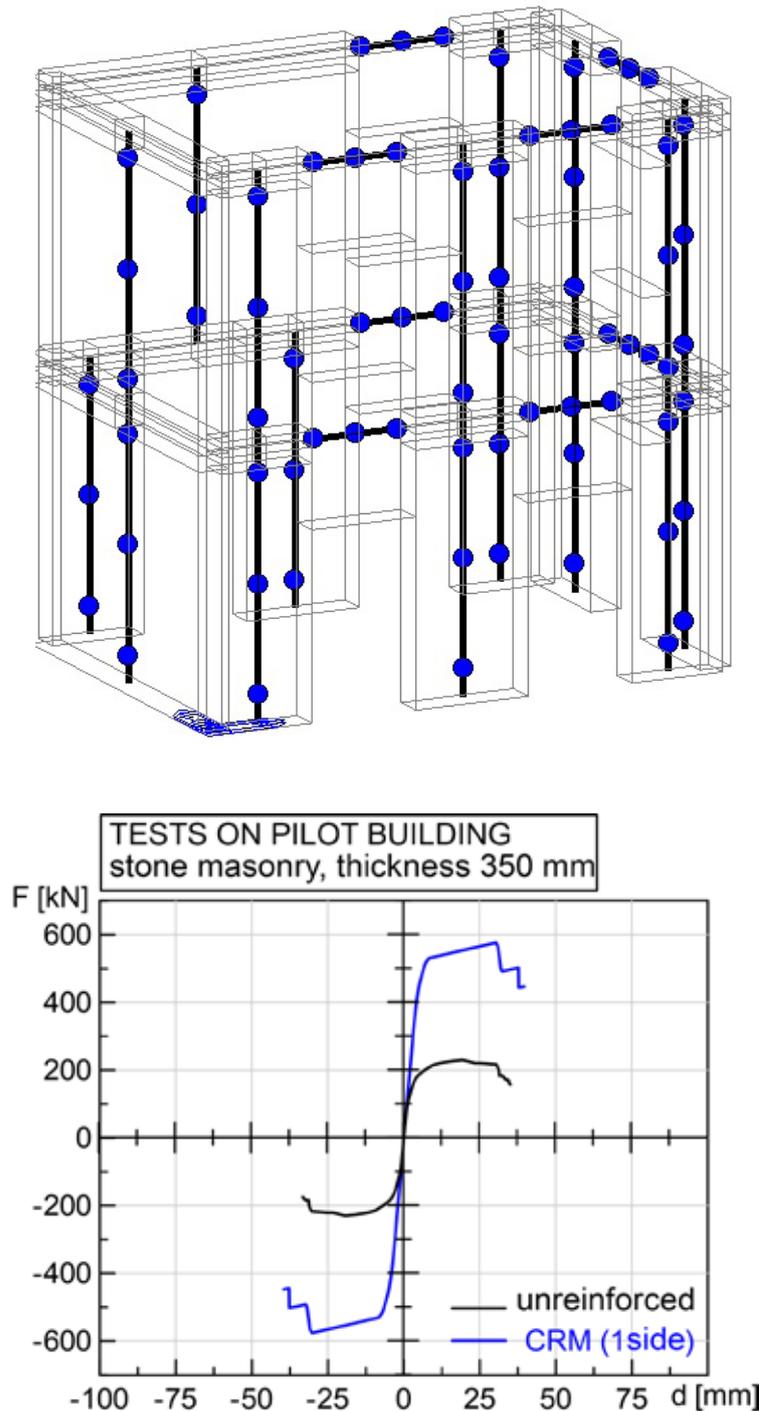
Fig.30 - Numerical simulations on the pilot building with the lumped plasticity model (LPM): (a) overall view of the model and (b) capacity curves.

For the evaluation of the overall seismic performance of existing masonry buildings, the practice often adopts the method of lumped plasticity model (LPM, **Fig.29c**). It involves schematizing the structure using one-dimensional beam elements with elastic behavior, representing the resistant macro-elements (piers and spandrels), connected through rigid nodes. The nonlinear behavior of piers and spandrels is condensed into localized plastic hinges, which account for different collapse mechanisms (such as diagonal cracking or flexure).

To adapt the **LPM** model for simulating masonry reinforced with CRM, the characteristics of plastic hinges were calibrated in terms of strength and displacement capacity, using the results of simulations performed with the MLM model on structural elements. The comparison between the results of LPM and MLM models applied at a large scale (pilot building - **Fig.30**) validated the LPM method for the global analysis of entire structures **[28]**.

5

PRACTICAL RULES FOR DESIGN

5.1 METHODOLOGICAL APPROACH TO STRUCTURAL CONSOLIDATION DESIGN

One of the main objectives of structural engineering design is to define structural interventions for both new and existing buildings that reduce the risk associated with potential seismic events, generally in accordance with specific rules: the so-called technical standards for constructions (NTC). One of the key principles of the new generation standards is the performance-based concept, which in seismic context primarily involves conferring ductility to structural elements and, more generally, to the structure as a whole. In more detail, this concept can be expressed as the ability to ensure certain levels of safety in the face of significant displacements, allowing the structure to undergo localized rather than generalized damage. By directly ensuring this capacity to the structure (through new design) or indirectly (using appropriate consolidation systems in the case of existing constructions), it is able to perform work and, consequently, dissipate energy.

Once the minimum safety levels that the structure must guarantee are established, the designer defines the structural elements using various methods provided within the technical standards. In particular, regarding existing masonry structures, one of the often-used methods is the tabular method.

This method associates an improvement coefficient of the mechanical performance of reinforced masonry with a masonry type and reinforcement technique. Being a general method, it must ensure normative safety in all possible conditions. However, the application of this method generally leads the designer to underestimate the capacity of the structures, in exchange for simplifying calculations and reducing computational burdens.

Another fundamental principle of standards and design activity is the scientific method based on the following stages:
▪ observation of the phenomenon;
▪ formulation of hypotheses;
▪ execution of experimental tests;
▪ data analysis;
▪ development of a predictive analytical model.

This approach allows for a specific evaluation of the current state as well as the improvements achievable through the application of a reinforcement technique, verifying them through specific tests on the structure. The difference in design approach applied to the case of an existing masonry structure can be highlighted as follows.
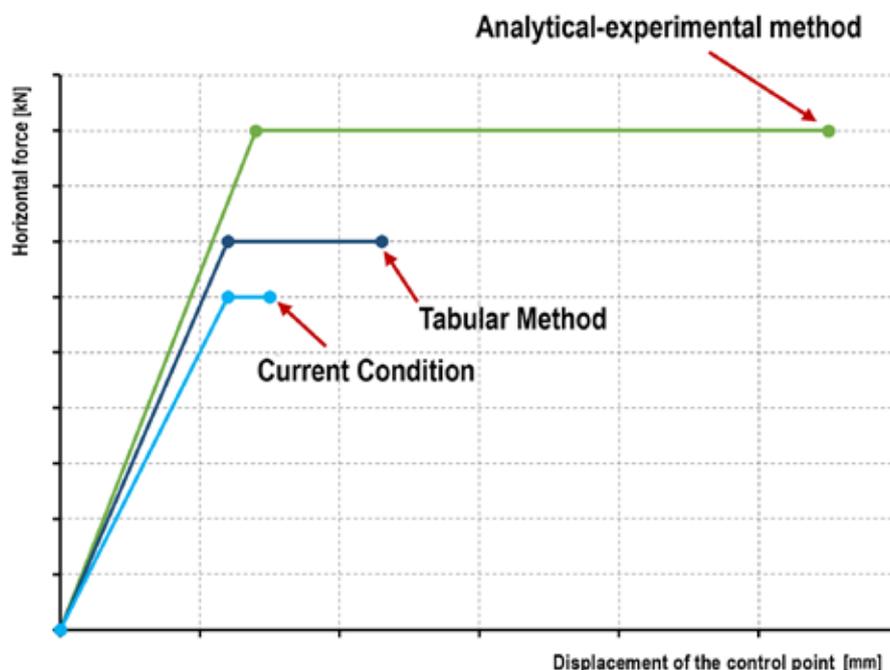
Tabular method vs. Data-supported analysis

The purpose of this chapter is to translate the substantial experimentation activity carried out and briefly outlined in the previous chapters into useful design principles for the professional. The analytical-experimental method proposed here, which relies on the database derived from this activity, allows for a specific approach to masonry and design intervention, making the best use of available information on the building, materials, and construction techniques. This represents the novelty, furthermore widely supported and validated, compared to a more generic and standardized, and therefore more "conservative," approach such as that proposed by table C8.5.II. of the 2018 NTC (tabular method).

The new methodological approach to consolidation design with the CRM System by Fibre Net, validated even more clearly after the results on full-scale buildings (as a

complement to the Push 'O Ver and CONSTRAIN Projects), merges the contents of the updated Design Guidelines with ZAG in 2023, where the main experimental results have been translated into technical bases and design rules, and the principle described in Eurocode 8 - Design of structures for seismic resistance - of "design assisted by testing".

This approach involves the use of experimental results, subject to processing, verification calculation, and validation, to design structural interventions where adequate calculation models are not available and it is necessary to use a large number of similar components essentially, confirming through control checks the hypotheses formulated in the design phase, taking into account the statistical uncertainty due to a limited number of tests.
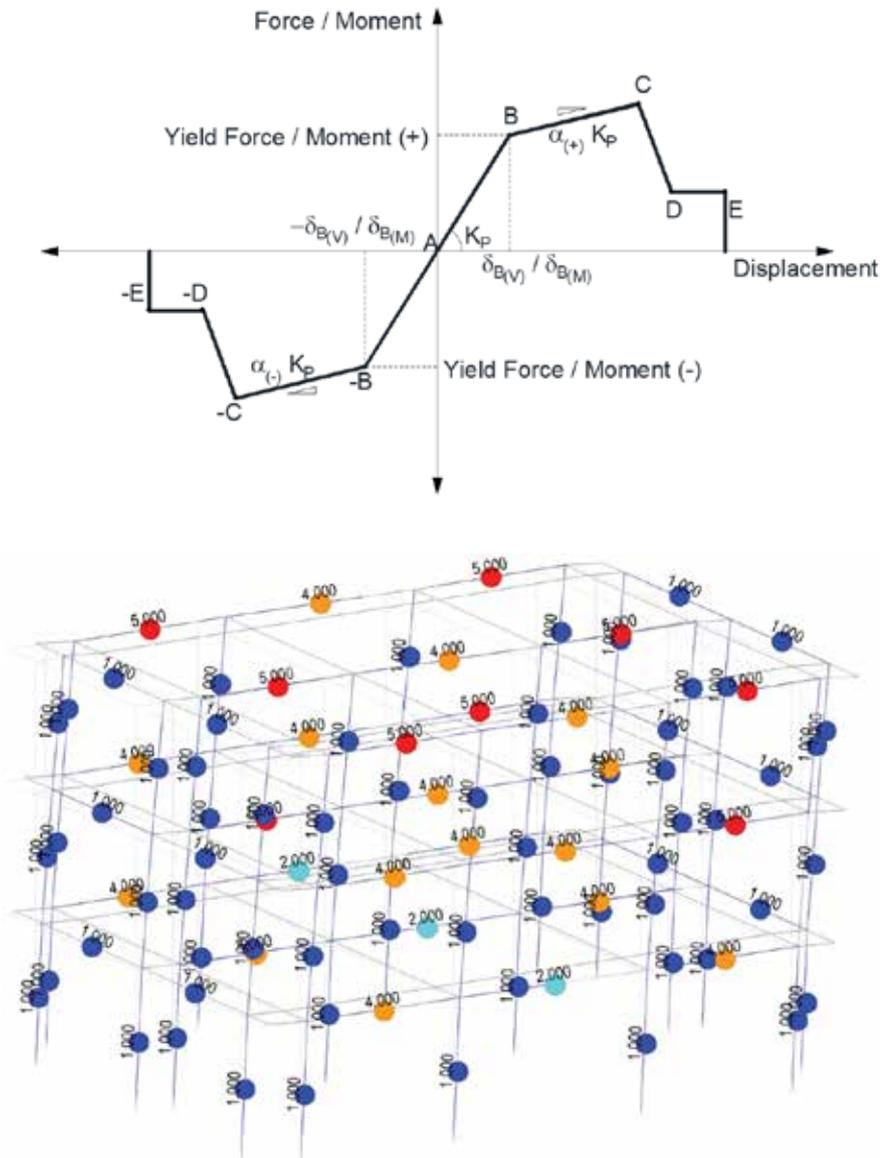
Fig.31 - *Simplified behavior of structural elements: from the single plastic hinge to its placement in a macroscopic model*

While the contents of the CRM Design Guideline allow the Designer to fully exploit the combinations of System components based on the strength and deformation capacity of the consolidated element, as well as to appreciate a significant reduction in seismic action through a structural factor (a generalistic approach) or equivalent viscous damping coefficients (a specific approach based on physical principles) at higher values, the "design assisted by testing", with hundreds of experimental tests conducted by the company, can fill some design gaps, especially in terms of deformation, not included in the ZAG Guideline. Therefore, by carefully addressing the design and studying the arrangement of construction details, the effectiveness of the CRM System is maximized: there is indeed a significant transformation from less ductile and essentially unpredictable elements (such as those of unreinforced masonry walls) to much more predictable behavioral elements, with defined collapse mechanisms reaching resistance capacity based on all CRM System components as well as ductile ones, with ultimate rotations relative to the chord for piers and spandrels, making them extremely more efficient in the presence of seismic action.

This design approach, with benefits both in terms of the load-bearing capacity of masonry structural elements and even more significant increases in terms of deformation for consolidated masonry, is, in conclusion, significantly more effective than one that uses simple amplification coefficients (tabular method) for improving the mechanical properties of reinforced masonry and, specifically, proves to be much more descriptive/ predictive of the actual behavior of the consolidated building.

Ductility vs Brittleness
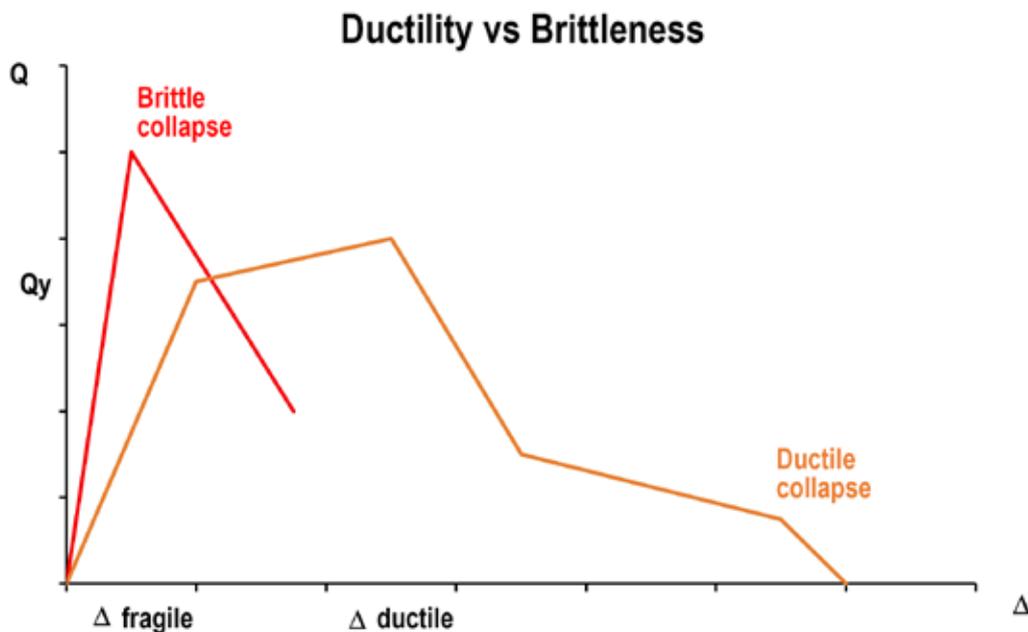
As already mentioned, the analytical method ("DESIGN ASSISTED BY TESTING"), compared to the tabular method, allows for precise reinforcement design, enabling the combination of effectiveness (building performance) and efficiency (costs), and/ or achieving higher safety levels with the same reinforcement. The application of the analytical method requires the availability of experimental data that concerns not only the reinforcement material but also the reinforcement applied to the individual structural element (piers and/or spandrels) and/or the entire reinforced structural unit subjected to shaking table test and/or push test.
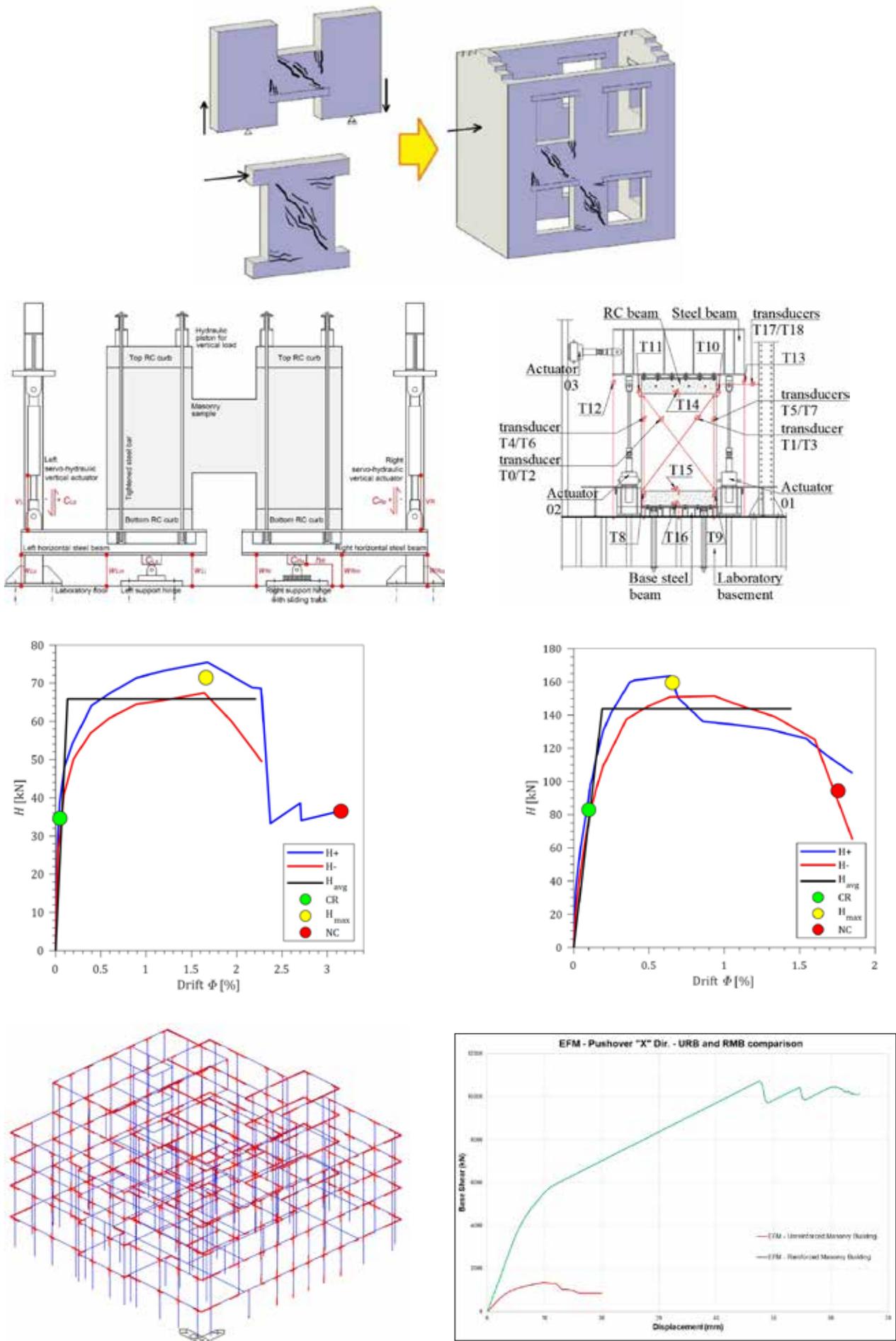
Fig.32 - *The structural model and the design assisted by testing: comparing the results between the existing condition and the design state of a masonry building*

5.2 PROJECT EXAMPLE

In order to highlight the differences in terms of effectiveness of various design approaches and to quantify the increase in displacement capacity and resistance of individual masonry structural elements, this chapter analyzes the behavior of a masonry building subjected to a design earthquake. The experimental background, robust data processing, and proposed formulations for defining the increase in displacement capacity and resistance - particularly in terms of displacement - are parameters on which the professional should rely to assess the quality and effectiveness of the consolidation system to be prescribed in the structural design.

To this end, the verifications of individual structural elements (piers and spandrels) and of the entire building will be presented in the following pages, along with verifications in the as-built configuration to facilitate reader understanding, performed according to different design approaches:
▪ NTC 2018 approach, Chapter 8;
▪ Analytical/experimental approach, Fibre Net formulation from 2015;
▪ Analytical/experimental approach, design by testing (from CONSTRAIN Project) by Fibre Net from 2023.

5.2.1 GEOMETRY, MATERIALS, AND LOAD DEFINITION

The building is three stories above ground with an overall height of 9.5 meters and has plan dimensions of 14.0m x 11.0m. The structure consists of five walls, three in the longitudinal direction and two in the transverse direction, as shown in the following figure. The three longitudinal walls are defined to have a different distribution of openings on the ground floor, while they are identical on the upper floors; the two transverse walls are symmetrical to each other. Each level has a height of 3.0 meters.

All walls have a constant thickness of 35 cm, made of stone masonry. The mechanical characteristics of the masonry have been deduced from experimental tests, as reported in Table 1. The floors consist of wooden beams and a 5 cm reinforced concrete slab. The presence of the slab ensures the presence of an effective diaphragm at each floor level. The roof is made up of wooden beams and planking.
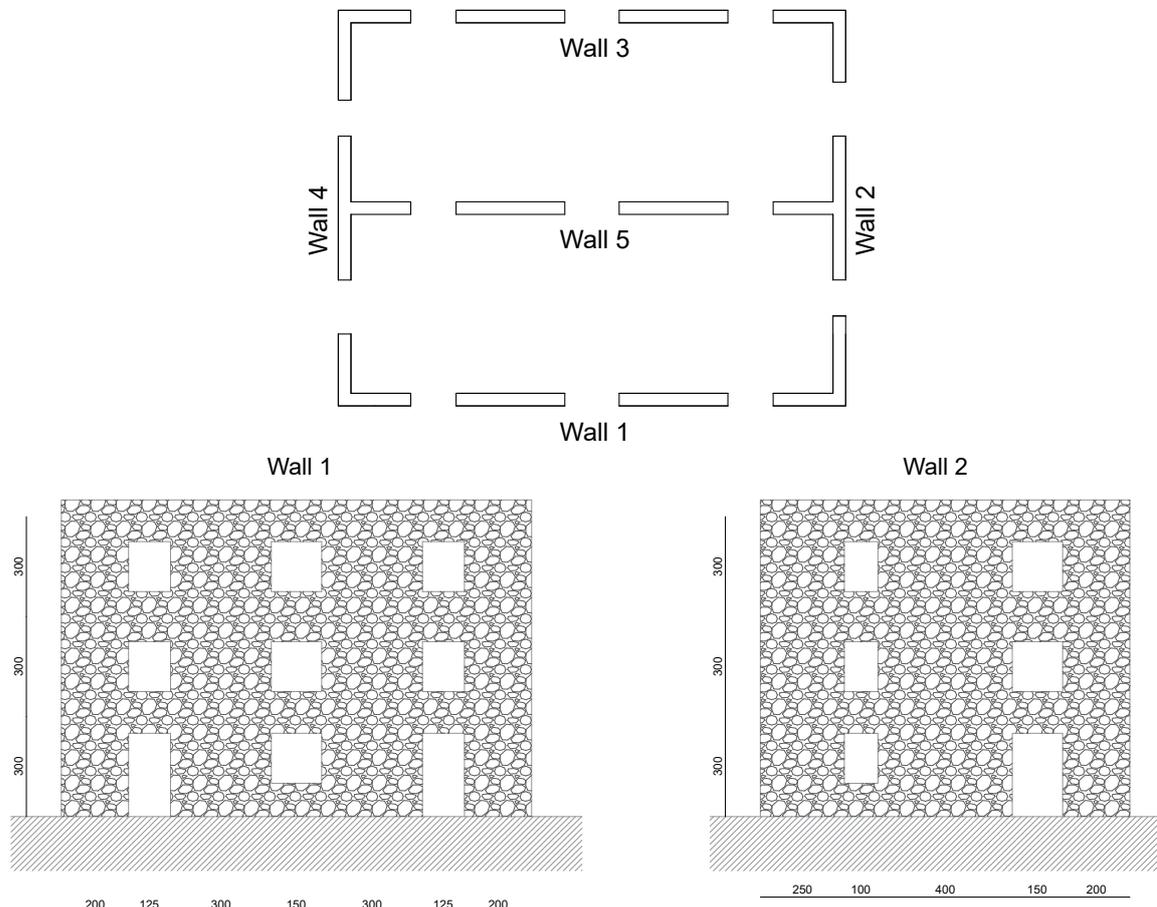
Fig.33 - *Geometry of the building and the two reference walls (longitudinal and transverse*

MATERIALS

Masonry

The stone masonry has the following mechanical properties:

Table 1 *- Mechanical properties of the masonry*

Stone masonry		
Compression strength ($f_{m;U}$)	2,48	MPa
Elastic modulus (E_m)	537	MPa
Ultimate strain (ε_{mu})	2,00	%
Shear strength ($\tau_{0,U}$)	0,08	MPa
Unit weight	21,0	kN/m^3

In the assessment of the existing condition, a unity confidence factor is considered. For a knowledge level LC3, extensive knowledge of the structure's geometry and construction details, as well as comprehensive knowledge of the mechanical properties of the materials, must be ensured. Additionally, material safety factors are applied in the evaluation of design strengths.

Table 2 *- Design Mechanical Characteristics*

Stone masonry – design strengths		
Compression strength (f_m)	1,24	MPa
Shear strength (τ_0)	0,04	MPa

CRM System

The reinforcement is implemented by applying the CRM System on both sides of the masonry.

The mechanical characteristics of the system components are listed in the following table.

Table 3 *- Mechanical characteristics of the reinforcement*

Mortar (M15)		
Compression strength (f_{cmo})	15,0	MP$_a$
Tensile strength ($f_{t,c}$)	3,0	MP$_a$
Elastic modulus (E_c)	9000	MP$_a$
GFRP mesh 66x66T96		
Mesh size	66x66	mm
Average tensile strength ($f_{tm,v}$)	495	MP$_a$
Nominal fiber area	8,9	mm^2
Elastic modulus (E_f)	25000	MP$_a$
Ultimate strain (ε_{fu})	1,45	%

To define the mechanical characteristics of the reinforced masonry, equivalent stiffness and self-weight are considered, calculated as a weighted average over the thickness of the masonry and reinforcement.

$$E_{eq} = \frac{E_m\, t_m + E_c\, t_c}{t_m}$$

Characteristics of reinforced masonry		
Equivalent elastic modulus (E_{eq})	2080	MPa
Equivalent weight (W_{eq})	24	kN/m^3

LOAD DEFINITION

The applied loads are listed in the following tables.

Floor loads

Table 4 - *Loads at floor level*

Permanent loads	**G₁** Concrete slab (t = 5 cm) Wood planking (t = 3 cm) Wooden beams (16x24 cm)	**1,85** 1,25 0,18 0,42	**kN/m²** kN/m² kN/m² kN/m²
	G₂ Tamping Flooring Non-structural slab Vapor barrier	**2,41** 1,20 0,20 1,00 0,01	**kN/m²** kN/m² kN/m² kN/m² kN/m²
Accidental load	**Q** (Tab 3.1.II NTC 2018 – residential use environment)	**2,00**	**kN/m²**

Roofing

Table 5 - *Loads in roofing*

Permanent loads	**G₁** Wood planking (t = 3 cm) Wooden beams (12x20 cm, i=70 cm)	**0,39** 0,18 0,21	**kN/m²** kN/m² kN/m²
	G₂ Roofing tiles Ventilation layer Waterproofing Thermal insulation Vapor barrier	**1,20** 0,80 0,15 0,04 0,20 0,01	**kN/m²** kN/m² kN/m² kN/m² kN/m² kN/m²
Accidental load	**Q** (Snow)	**1,20**	**kN/m²**

Seismic action

Table 6 - *Seismic parameters*

Nominal life	50 years
Use coefficient (c_u)	1
Subsurface category	B
Topographic category	T1
a_g/g	0,1909
F_0	2,44
T_C^*	0,33
S_S	1,20
C_C	1,373
S_T	1,00

5.2.2 COMPARISON BETWEEN FORMULATIONS FOR THE CALCULATION OF THE RESISTANCE CAPACITY

After completing the seismic analysis of the building, the obtained results on some of its structural elements are evaluated. The stresses, resistances, and post-elastic behavior will serve as a comparison term at the local level of each individual element within the building, among various methodological verification approaches. These aspects will then be encompassed in the global seismic response.
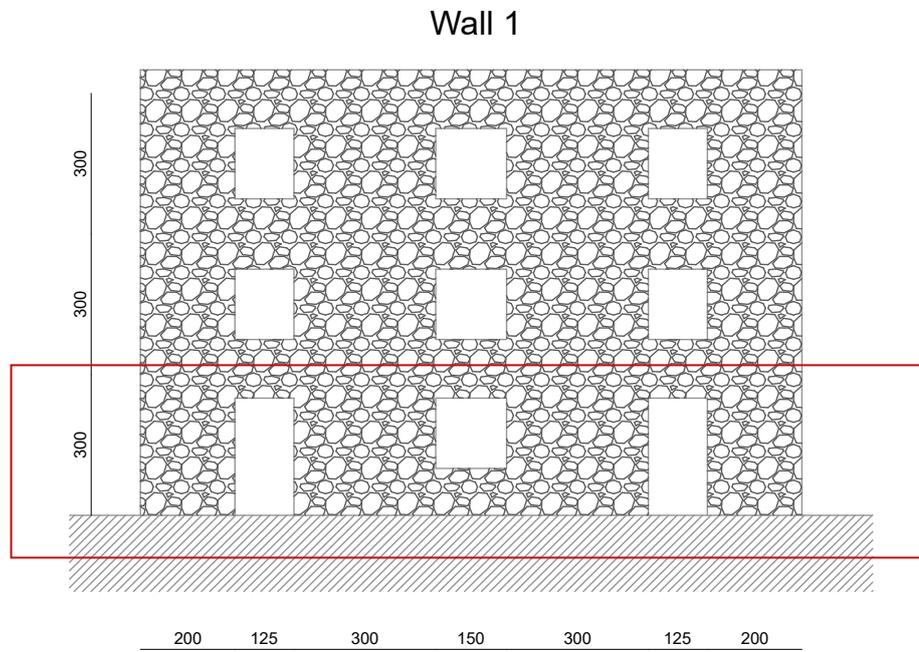
Fig.34 - *Detail of the analyzed wall.*

So the focus is on four masonry piers, each with different dimensions. The resistance capacity of the unreinforced elements is evaluated, serving as a starting point for comparing the various formulations available to the designer for calculating the resistance capacity of the elements reinforced with the CRM RI-STRUTTURA System by Fibre Net.

Table 7 - *The geometry of the analyzed wall.*

	Spessore	Lunghezza b	Altezza h	Area A	Inerzia J	Rigidezza k
	[m]	[m]	[m]	[mm²]	[mm⁴]	[N/mm]
Setto 1	0.35	2.00	2.88	7.000E+05	2.333E+11	2.196E+04
Setto 2	0.35	3.00	2.56	1.050E+06	7.875E+11	4.790E+04
Setto 3	0.35	3.00	2.56	1.050E+06	7.875E+11	4.790E+04
Setto 4	0.35	2.00	2.88	7.000E+05	2.333E+11	2.196E+04

EVALUATION OF THE CAPACITY IN THE CURRENT STATE.

The resistance of the individual masonry pier is evaluated by considering the weakest mechanism among the following:

- Bending

$$M_{Rd,bend} = \frac{\sigma_0\, b^2\, t}{2}\left(1 - \frac{\sigma_0}{0.85\, f_m}\right)$$

$$V_{Rd,bend} = \frac{2 \cdot M_{Rd,bend}}{h}$$

- Diagonal cracking

$$V_{Rd,df} = \frac{1.5\, \tau_0\, b\, t}{h/b}\sqrt{\left(1 + \frac{\sigma_0}{1.5\, \tau_0}\right)}$$

- Shear-sliding

$$V_{Rd,sc} = \frac{1.5\, \tau_0 + 0.4\sigma_0}{1 + \dfrac{3\, h_0 \tau_0}{b\, \sigma_0}}\, b\, t$$

Table 8 - *Masonry resistance in the current state*

	Vertical loads	σ_0	$V_{Rd,min}$ (=$V_{Rd,sc}$)	δ_e	δ_u
	[kN]	[N/mm^2]	[kN]	[mm]	[mm]
Septum 1	213.10	0.30	65.91	3.00	14.40
Septum 2	332.45	0.32	111.47	2.33	12.80
Septum 3	332.45	0.32	111.47	2.33	12.80
Septum 4	213.10	0.30	65.91	3.00	14.40
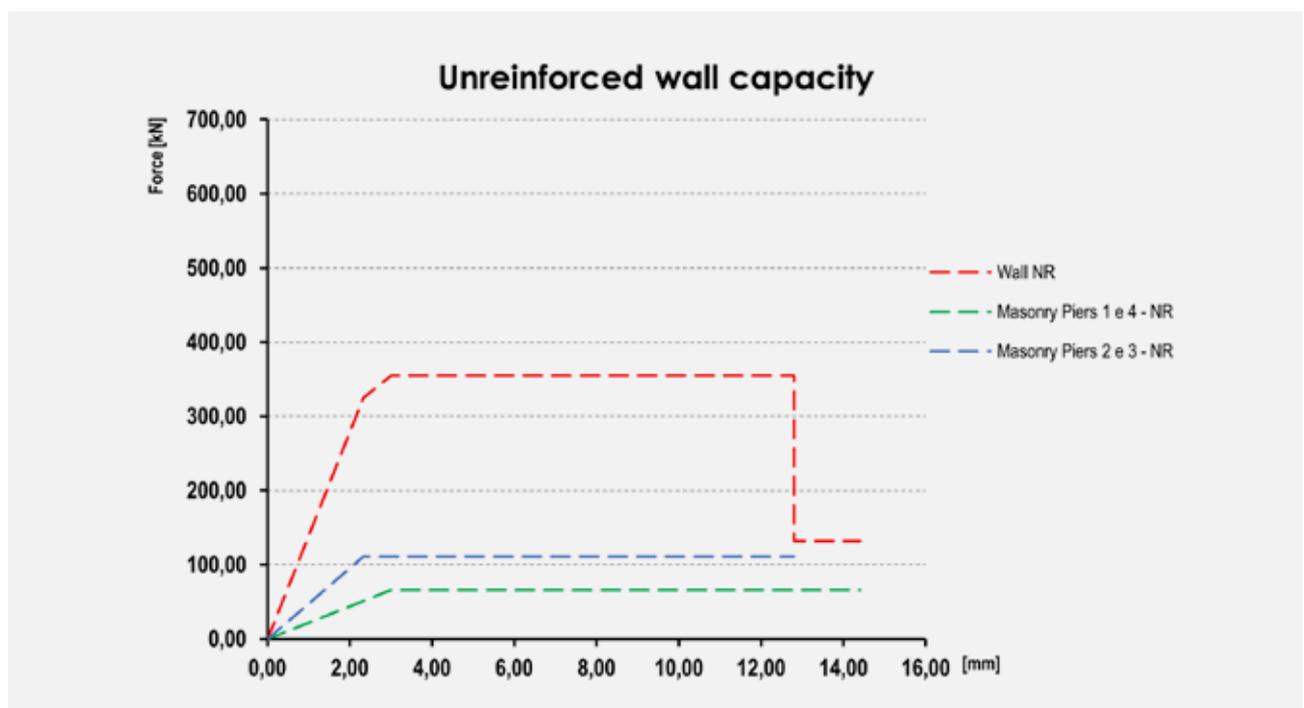
EVALUATION OF THE CAPACITY IN THE DESIGN STATE

In this chapter, the results obtained by verifying the resistant elements according to the previously defined approaches are reported. To appreciate the differences between them, the results are expressed in terms of displacement capacity and strength, parameters, especially the first of the two, necessary for identifying a very effective consolidation system.

Capacity according to NTC 2018 Table C8.5.II

This verification approach, evaluating the resistance capacity of the element consolidated with the CRM System - considered as reinforced plaster - follows the contents of Chapter 8 of the NTC 2018, particularly those of Table C8.5.II [20], [33]: while amplification coefficients are inserted for the calculation of the resistances and elastic moduli of the consolidated material (specifically a coefficient of 1.5), no amplifications are specified for defining the displacement behavior of the reinforced elements.

Table 9 - *Mechanical properties of masonry*

Stone masonry		
Compressive strength ($f_{m,U}$)	3,70	MPa
Elastic modulus (E_m)	805	MPa
Ultimate deformation (ε_{mu})	2,00	%
Shear strength ($\tau_{0,U}$)	0,12	MPa
Volume weight	21,0	kN/m^3
Stone masonry – design strengths		
Compressive strength (f_m)	1,85	MPa
Shear strength (τ_0)	0,06	MPa

Considering the same formulations as the previous paragraph, we obtain:

Table 10 - *Strength of reinforced masonry (Table C8.5.II NTC 2018)*

	Vertical loads	σ_0	$V_{Rd,min}$ (=$V_{rd,sc}$)	δ_e	δ_u
	[Kn]	[N/mm^2]	[kN]	[mm]	[mm]
Septum 1	213.10	0.30	74.08	2.25	14.40
Septum 2	332.45	0.32	129.56	1.80	12.80
Septum 3	332.45	0.32	129.56	1.80	12.80
Septum 4	213.10	0.30	74.08	2.25	14.40
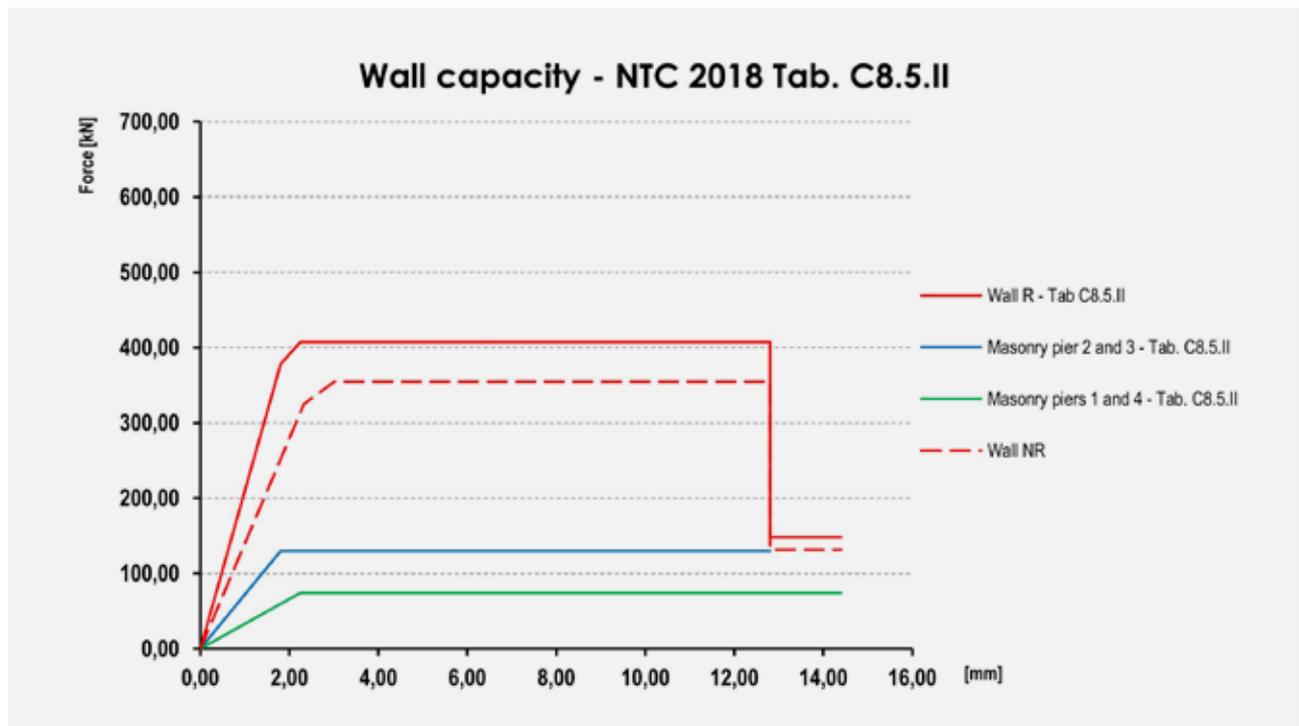
Capacity according to experimentation - formulation 2015

Following this verification approach, now outdated and subsequent to the processing of data obtained from the first important campaigns of diagonal compression, the evaluation of the resistance capacity of the element consolidated with the CRM Ri-Struttura System follows the formulations inserted in Gattesco, Boem **[09]**, in which the resistant contributions of the individual components of the System are considered, appropriately scaled to differentiate the type of masonry and its thicknesses. Since the initial research aimed to quantify effectiveness in terms of resistance, the important issue of displacement capacity was not addressed in detail: therefore, even in this case, despite a significant increase in resistance, no amplification is specified for defining the deformative behavior of the reinforced elements.

Table 11 - Mechanical properties of masonry **[28]**

Stone masonry			
Compressive strength ($f_{m,R}$)	The strength of masonry is amplified by 20% $f_{m,R} = 1,20\, f_m$	2,98	MPa
Elastic modulus (E_m)	It is calculated as a weighted average over the thickness 2080 MPa	2080	MPa
Ultimate deformation (ε_{mu})		2,00	%
Shear strength ($\tau_{0,U}$)	It is calculated based on the following formulation: $$\tau_{0\,(R)} = \beta \cdot \left(\tau_{0\,(U)} + m \cdot \frac{t_c}{t} \cdot \frac{f_{t,c}}{1.5} \right)$$ Defined through diagonal compression tests	0,21	MPa
Volume weight	It is calculated as a weighted average based on the thickness of the masonry and the reinforcement	23,0	kN/m³
Stone masonry – design strengths			
Compressive strength ($f_{m,d}$)		1,49	MPa
Shear strength (τ_0)		0,11	MPa

Considering the same formulations as the previous paragraph, adopting the values from the tables above, we obtain

Table 12 - *Masonry mechanical properties*

	Vertical loads	σ_0	$V_{Rd,min}$ (=$V_{rd,sc}$)	δ_e	δ_u
	[kN]	[N/mm²]	[kN]	[mm]	[mm]
Septum 1	213.10	0.30	87.60	0.87	14.40
Septum 2	332.45	0.32	162.77	0.70	12.80
Septum 3	332.45	0.32	162.77	0.70	12.80
Septum 4	213.10	0.30	87.60	0.87	14.40
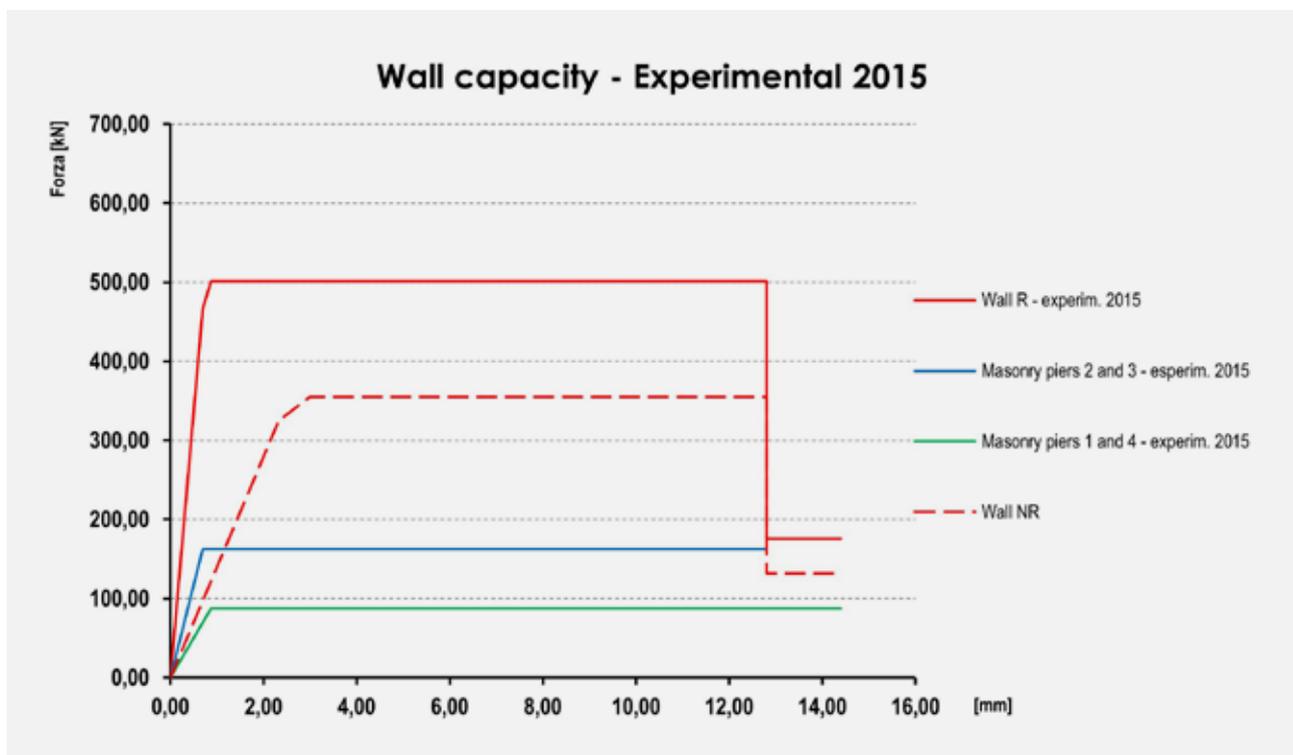
Using this last formulation, we get:

Table 13 - *Strength of reinforced masonry*

Wall strength		
Reinforced wall strength:	500.74	kN
Unreinforced wall strength:	292.74	kN
Ratio between resistances:	1.71	

Capacity according to experimentation - CONSTRAIN project results

Finally, following this verification approach, defined following a significant data elaboration from the CONSTRAIN Project and retrospectively, under a different perspective, from previous experimental projects, the evaluation of the resistance capacity and the displacement capacity of the element consolidated with the CRM RI-STRUTTURA System follows the considerations expressed in the Final Report of the Project (2023) **[23]**. In this case, besides assessing the strength of individual elements according to various collapse mechanisms (also dependent on the type of reinforcement mesh used), there are considerations and indications regarding the deformation behavior of the same. By observing the behavior of the unreinforced and reinforced elements, one can note the significant difference in energy dissipation capacity between the two elements: this aspect is crucial when dealing with a consolidation project.

Table 14 - Masonry mechanical properties

Stone masonry			
Compressive strength ($f_{m,R}$)	$f_{m,R} = f_{m.U}$	2,48	MPa
Elastic modulus (E_m)	It is calculated as a weighted average over the thickness 2080 MPa	2080	MPa
Ultimate deformation (ε_{mu})		2,00	%
Shear strength ($\tau_{0,U}$)	It is calculated based on the following formulation: $\tau_{0(R)} = \beta \cdot \left(\tau_{0(U)} + m \cdot \frac{t_c}{t} \cdot \frac{f_{t,c}}{1.5} \right)$ Defined through compression tests diagonal	0,21	MPa
Volume weight	It is calculated as a weighted average based on the thickness of the masonry and the reinforcement	23,0	kN/m³
Stone masonry – design strengths			
Compressive strength (f_m)		1,24	MPa
Shear strength (τ_0)		0,11	MPa

The results obtained by adopting a mesh with dimensions of 66x66 are summarized below:

Table 15 - Masonry mechanical properties

	Vertical loads	σ_0	$V_{Rd,min}$ (=$V_{rd,bend}$)	δ_e	0.80 V_{Rd}	δ_u
	[kN]	[N/mm²]	[kN]	[mm]	[kN]	[mm]
Septum 1	213.10	0.30	163.81	1.389	131.05	72
Septum 2	332.45	0.32	406.51	1.495	325.21	64
Septum 3	332.45	0.32	406.51	1.495	325.21	64
Septum 4	213.10	0.30	163.81	1.389	131.05	72

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Wall capacity - Experimental CONSTRAIN with mesh 66x66
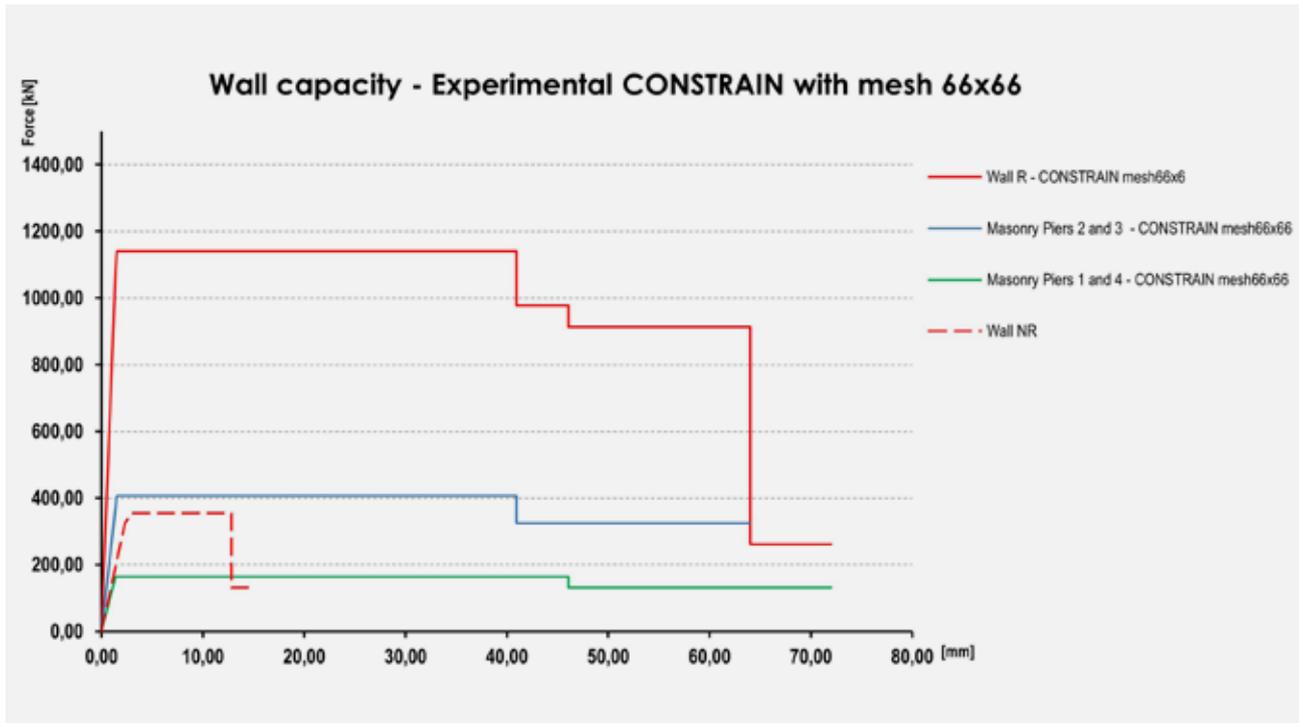
Using this last formulation, we obtain:

Table 16 - *Resistenza muratura rinforzata*

Wall strength		
Reinforced wall strength:	1140.64	kN
Unreinforced wall strength:	292.74	kN
Ratio between resistances	3.90	

Similarly, the resistance capacity can be evaluated with the application of the CRM System using a mesh size of 33x33. This calculation methodology allows for the utilization of a larger quantity of mesh to further enhance the resistance.

Table 17 - *Strength of reinforced masonry*

	Vertical loads	σ_0	$V_{Rd,min}$ (=$V_{rd,bend}$)	δ_e	0.80 V_{Rd}	δ_u
	[kN]	[N/mm^2]	[kN]	[mm]	[kN]	[mm]
Septum 1	213.10	0.30	206.59	1.751	165.27	72
Septum 2	332.45	0.32	525.56	1.933	420.44	64
Septum 3	332.45	0.32	525.56	1.933	420.44	64
Septum 4	213.10	0.30	206.59	1.751	165.27	72
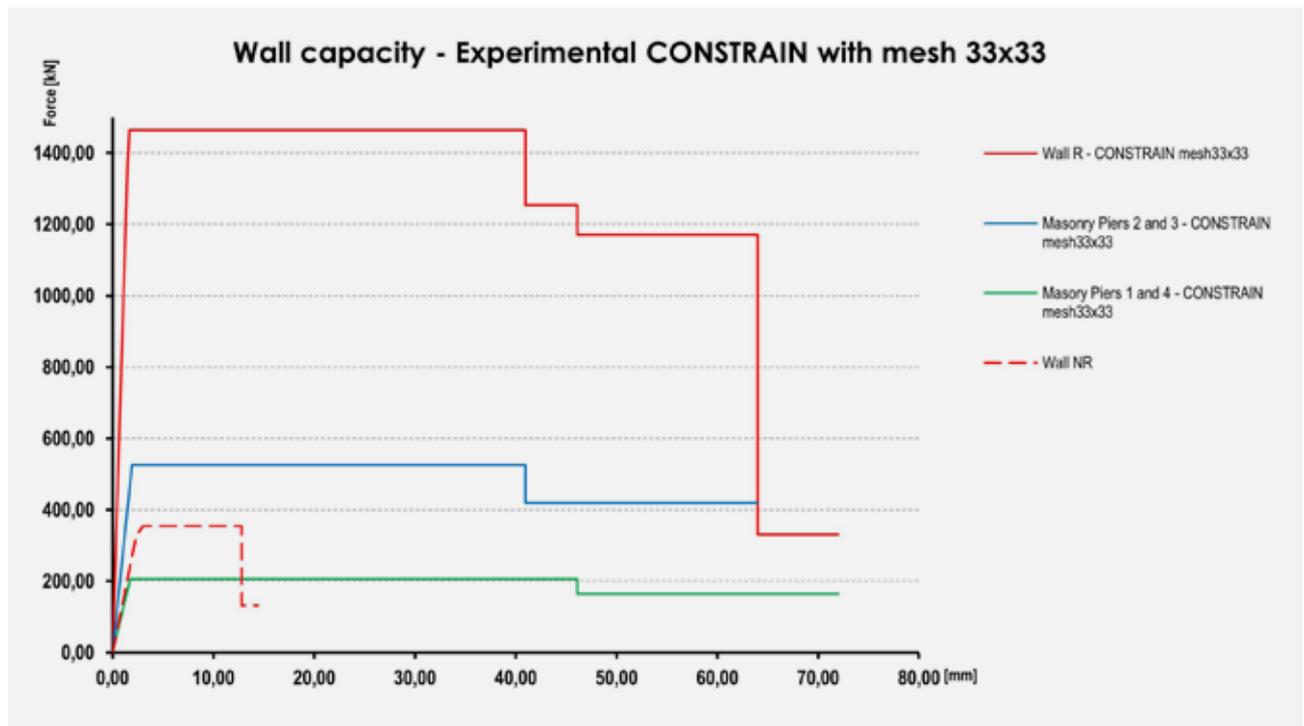
Using this last formulation, we obtain:

Table 18 - Strength of reinforced masonry

Wall strength		
Reinforced wall strength:	1464.28	kN
Unreinforced wall strength:	292.74	kN
Ratio between resistances	5.00	

5.2.3 CONCLUSIONS ON THE DESIGN APPROACH

In this chapter, the results obtained following different methodological verification aspects are summarized. Significant differences are noted already in the comparison between the indications according to NTC 2018 and those reported in the Fibre Net formulation of 2015. However, these differences are quantified exclusively in terms of the resistance of the consolidated element.

If the designer instead wishes to follow the Fibre Net design by testing approach after processing the results of the CONSTRAIN Project, they could also benefit from a significant contribution in terms of the deformative behavior of the applied reinforcement, significantly improving the behavior of the individual structural element and, as a consequence, of the entire building.

Conclusions - comparison of formulations
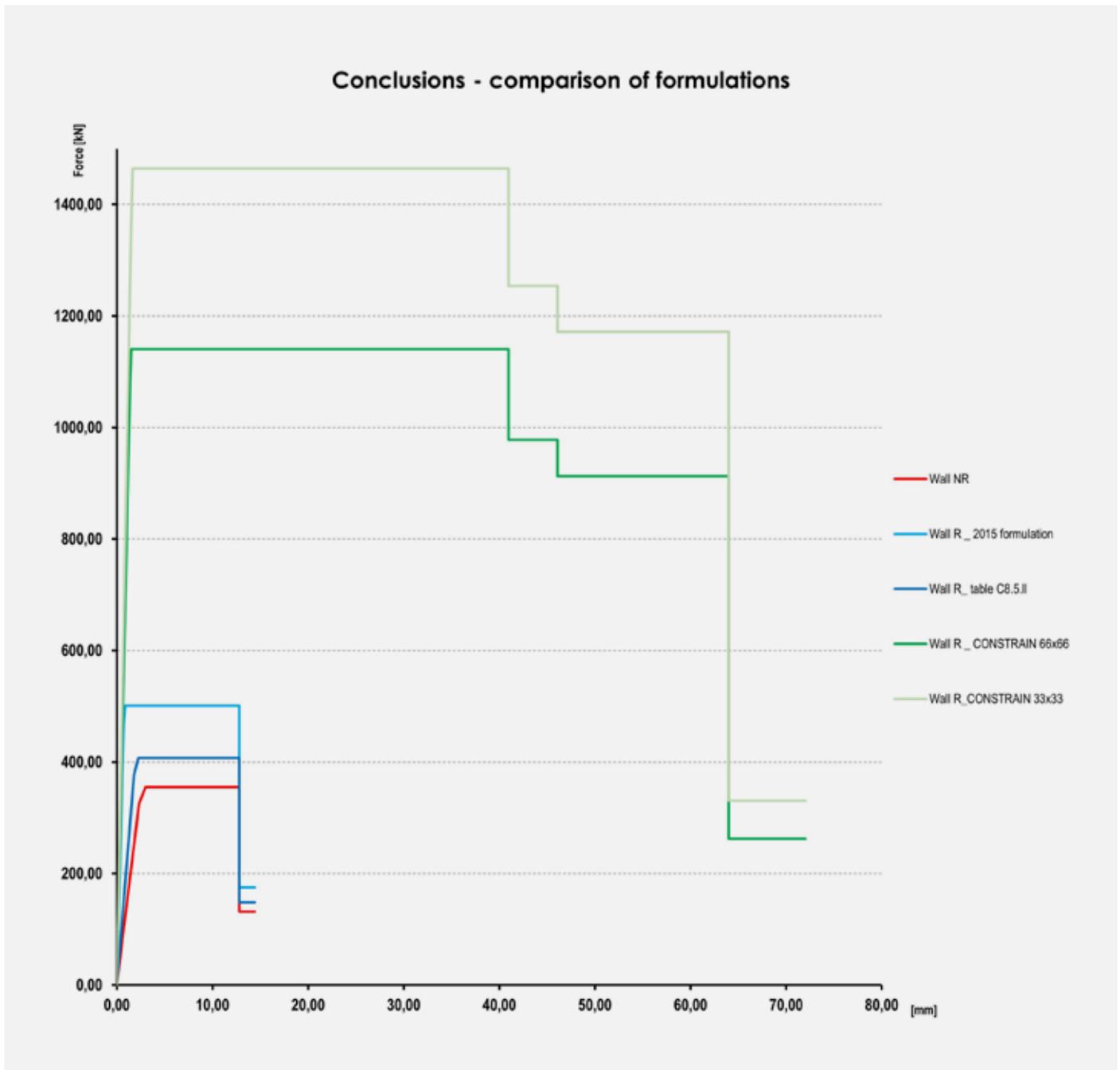
Numerical comparison between design approaches

	Vrd	δe	δu	ΔVrd	Δδu
Unreinforced element	**kN**	**mm**	**mm**	**%**	**%**
Pier 1	65.91	3.00	14.40	--	--
Pier 4	65.91	3.00	14.40	--	--
Pier 2	111.47	2.33	12.80	--	--
Pier 3	111.47	2.33	12.80	--	--
Wall	354.76	--	--	--	--
NTC 2018 Tab. C8.5.II	kN	mm	mm	%	%
Pier 1	74.08	2.25	14.40	12%	--
Pier 4	74.08	2.25	14.40	12%	--
Pier 2	129.56	1.80	12.80	16%	--
Pier 3	129.56	1.80	12.80	16%	--
Wall	407.28	--	--	15%	
Experimental 2015 Formulation	kN	mm	mm	%	%
Pier 1	87.60	0.87	14.40	33%	--
Pier 4	87.60	0.87	14.40	33%	--
Pier 2	162.77	0.70	12.80	46%	--
Pier 3	162.77	0.70	12.80	46%	--
Wall	500.74	--	14.40	41%	--
Experimental CONSTRAIN project (66x66 mesh)	kN	mm	mm	%	%
Pier 1	163.81	1.39	72.00	149%	400%
Pier 4	163.81	1.39	72.00	149%	400%
Pier 2	406.51	1.49	64.00	265%	400%
Pier 3	406.51	1.49	64.00	265%	400%
Wall	1140.64	--	--	222%	--
Experimental CONSTRAIN project (33x33 mesh)	kN	mm	mm	%	%
Pier 1	206.59	1.50	72.00	213%	400%
Pier 4	206.59	1.50	72.00	213%	400%
Pier 2	525.55	1.65	64.00	371%	400%
Pier 3	525.55	1.65	64.00	371%	400%
Wall	1464.28	--	--	313%	--

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5.2.4 NUMERICAL MODEL AND ANALYSIS

The reinforcement intervention on the entire building is proceeded with. The structure is modeled using Midas Gen 2023 software (v 1.1) **[30]**, employing equivalent frame schematization to conduct a modal dynamic analysis. Piers and spandrels are modeled using one- dimensional elements, adopting Dolce's formulation **[31]** to assess deformable sections.
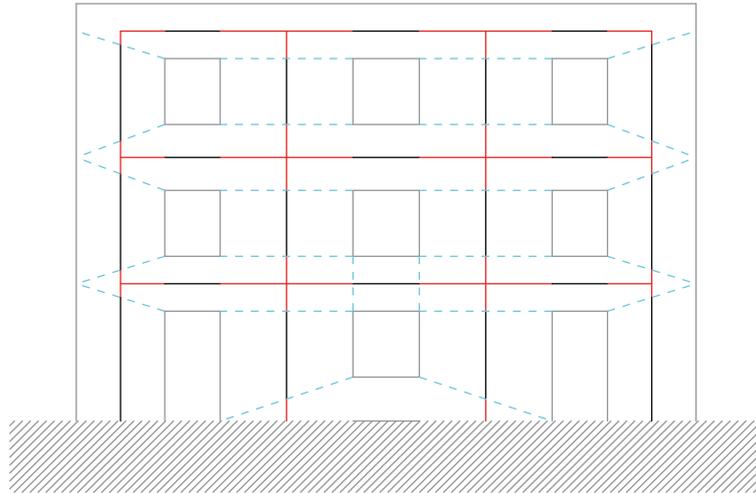
Fig. 35 - Equivalent frame modeling. Definition of effective heights.
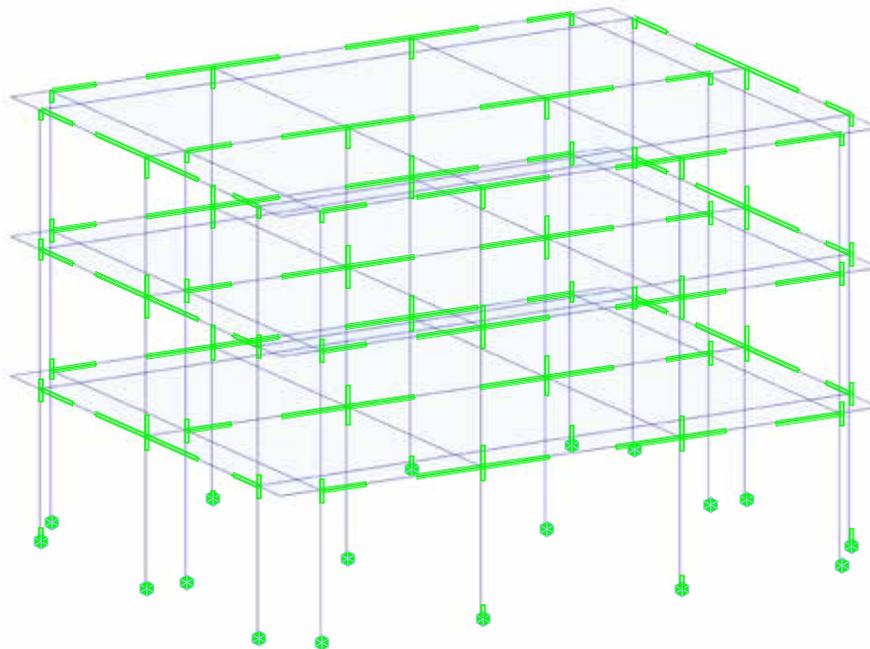
Fig. 36 - Numerical model with rigid elements and constraints.

The following nomenclature is established for the elements.

Wall 1

1-07-01	1-07-02	1-07-03
1-01-03　　1-02-03		1-03-03　　1-04-03
1-06-01	1-06-02	1-06-03
1-01-02　　1-02-02		1-03-02　　1-04-02
1-05-01	1-05-02	1-05-03
1-01-01　　1-02-01		1-03-01　　1-04-01

Wall 2

2-07-01	2-07-02
2-01-03　　2-02-03	2-03-03
2-06-01	2-06-02
2-01-02　　2-02-02	2-03-02
2-05-01	2-05-02
2-01-01　　2-02-01	2-03-01
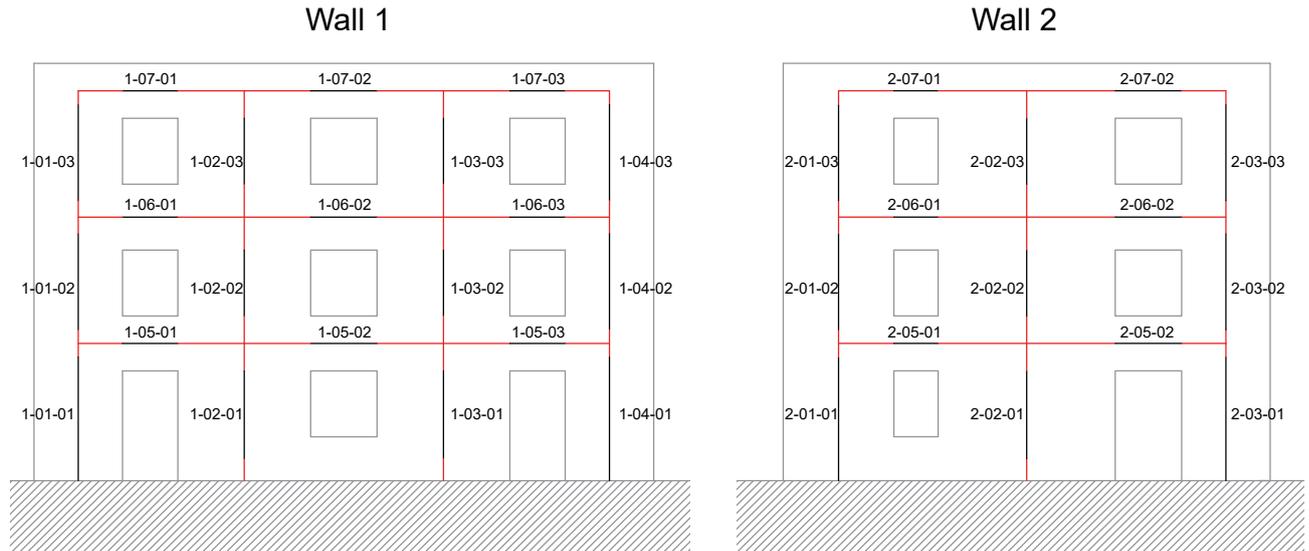
Fig. 37 - *Elements nomenclature*

In the numerical model, each masonry pier and each spandrel are defined in terms of material and section. Additionally, the floor diaphragm is defined at each horizontal level with reinforced concrete slab. Vertical loads are applied as linear loads on beams, while seismic loads are calculated based on the response spectrum (following what is reported in paragraph 5.2). Load combinations are defined as per NTC 2018, considering a 5% accidental eccentricity.

Dynamic behavior of the unreinforced structure

The first 20 vibration modes of the structure are reported in the following table.

Table 19 - *Modal response of the unreinforced structure, modeled as an equivalent frame.*

Mode	Frequency		Period	Tolerance
	(rad/sec)	(cycles/sec)	(sec)	
1	21.55	3.43	0.2916	0.00E+00
2	28.19	4.49	0.2229	0.00E+00
3	32.29	5.14	0.1946	0.00E+00
4	64.01	10.19	0.0982	1.04E-136
5	81.34	12.95	0.0772	6.29E-122
6	94.26	15.00	0.0667	3.15E-107
7	96.01	15.28	0.0654	1.18E-109
8	98.47	15.67	0.0638	4.06E-107
9	104.11	16.57	0.0603	8.70E-104
10	105.26	16.75	0.0597	8.17E-102
11	108.29	17.24	0.0580	2.12E-102
12	117.78	18.74	0.0533	8.56E-99
13	126.21	20.09	0.0498	3.27E-92
14	133.65	21.27	0.0470	8.95E-91
15	136.39	21.71	0.0461	7.02E-90
16	144.02	22.92	0.0436	5.39E-87
17	144.45	22.99	0.0435	1.49E-86
18	167.57	26.67	0.0375	1.42E-80
19	167.85	26.71	0.0374	5.52E-80
20	172.70	27.49	0.0364	5.41E-80

The shapes of the first three vibration modes, obtained from the numerical model, are reported below.

MODE I° – predominant Y
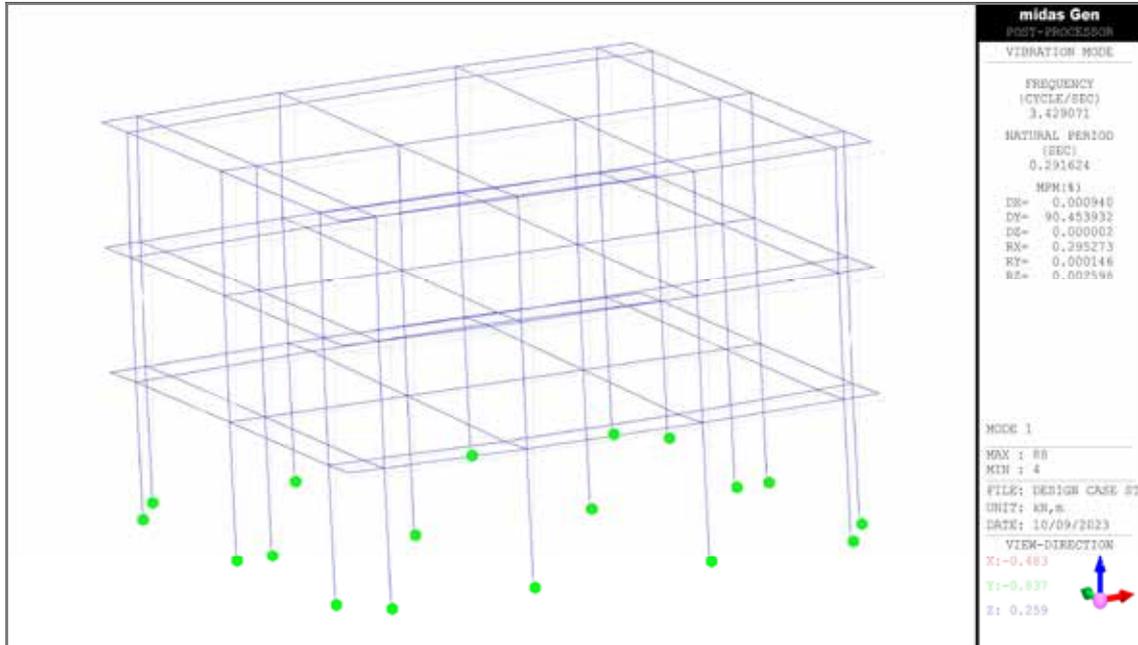
MODE II° – predominant X
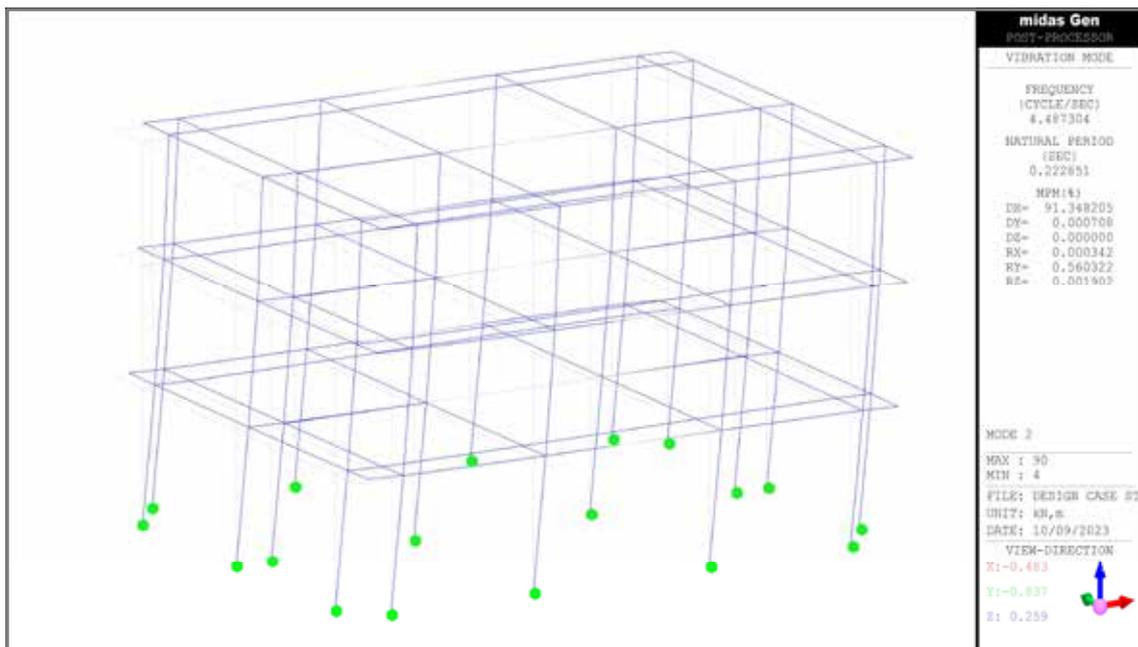
Dynamic behavior of the reinforced structure

The first 20 vibration modes of the structure are reported in the following table.

Table 20 - *Modal response of the reinforced structure, modeled as an equivalent frame.*

Mode	Frequency		Period
	(rad/sec)	(cycles/sec)	(sec)
1	31.97	5.09	0.1966
2	41.84	6.66	0.1502
3	47.75	7.60	0.1316
4	94.91	15.10	0.0662
5	120.26	19.14	0.0522
6	137.64	21.91	0.0457
7	141.93	22.59	0.0443
8	144.55	23.01	0.0435
9	153.56	24.44	0.0409
10	154.11	24.53	0.0408
11	158.85	25.28	0.0396
12	172.04	27.38	0.0365
13	184.29	29.33	0.0341
14	196.91	31.34	0.0319
15	200.68	31.94	0.0313
16	210.95	33.57	0.0298
17	213.20	33.93	0.0295
18	244.68	38.94	0.0257
19	245.12	39.01	0.0256
20	252.17	40.13	0.0249

The shapes of the first three vibration modes, obtained from the numerical model, are reported below.

MODE I° – predominant Y

MODE II° – predominant X
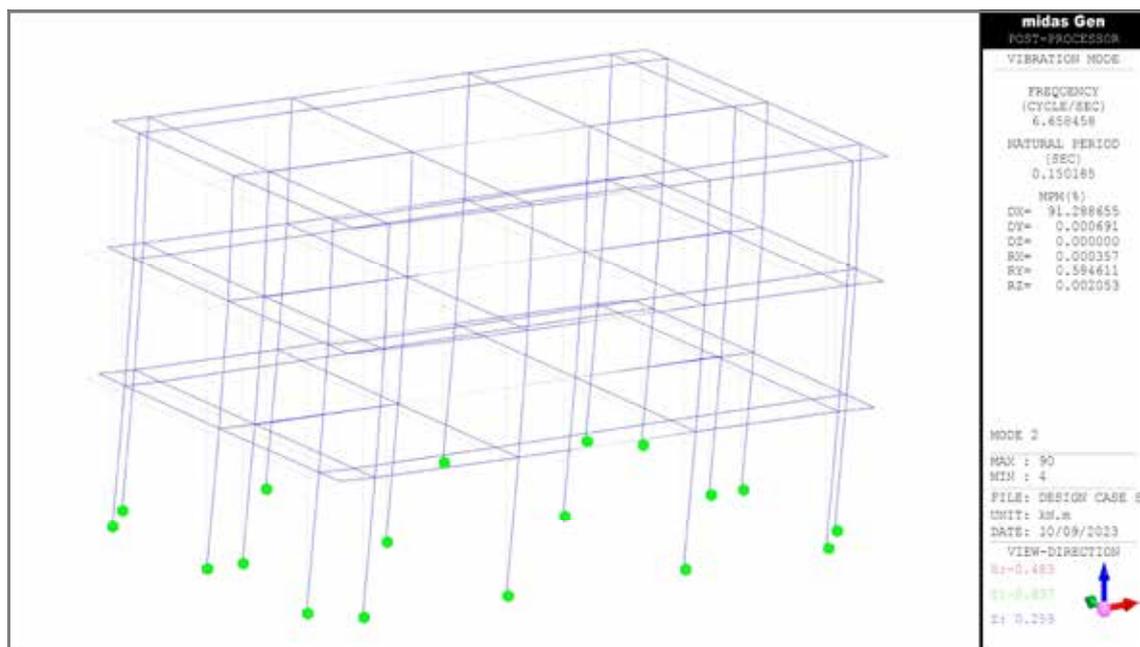
5.2.5 CHECKS OF THE UNREINFORCED STRUCTURE

The checks for wall 1 are reported, which are carried out considering the most severe combinations. Initially, local mechanisms are evaluated, as these are strongly influential on the behavior of the structure in-plane. Out-of-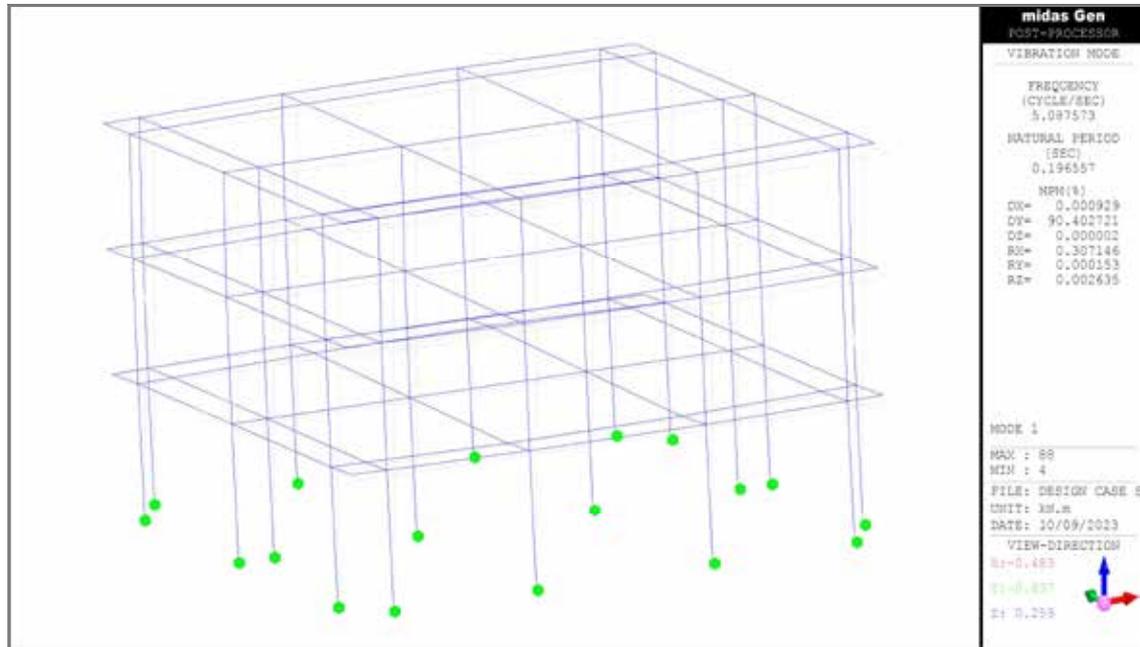plane resistance is assessed for piers 1-03-01, 1-03-02, 1-03-03. In-plane resistance is evaluated for all elements of the wall, taking the minimum value among resistance to diagonal cracking, sliding, and flexure.

Out-of-plane mechanism verification

The seismic action acting on the walls is calculated according to the procedure outlined. As indicated in point 7.2.3 of the NTC 2018, the horizontal seismic force acting on a wall can be assessed using equation 7.2.1.

$$F_a = \frac{S_a \cdot W_a}{q_a}$$

Where:
- S_a s calculated based on the ratio between the vibration period of the wall and the vibration period of the structure. The period of the wall is evaluated assuming the deformed shape of the first mode similar to a simply supported beam, with the following relationship:

$$T_a = 0.64\,h^2 \sqrt{\frac{A \cdot \gamma_m}{EI \cdot g}}$$

- W_a is the vertical load acting on the element
- q_a is the structural behavior factor determined for the specific case (in this case, equal to 1.5).

Wall_1 – Masonry Pier 3									
Pier	b (mm)	t (mm)	h (mm)	S_a	F_a (kN)	p (kN/m)	M_{sd} (kNm)	M_{Rd} (kNm)	M_{Rd}/M_{Sd}
10301	3000	350	2560	0.37	89.87	70.21	57.52	42.84	0.74
10302	3000	350	2000	0.44	58.94	58.94	29.47	28.65	0.97
10303	3000	350	2000	0.57	32.50	32.50	16.25	13.89	0.85

The elements are not verified.

In-plane mechanism verification

For in-plane checks, the following relationships are adopted.

Masonry piers

■ Flexure compression

$$M_{Rd,bend} = \frac{\sigma_0 \, b^2 \, t}{2} \left(1 - \frac{\sigma_0}{0.85 \, f_m} \right)$$

$$V_{Rd,bend} = \frac{2 \cdot M_{Rd,bend}}{h}$$

■ Diagonal cracking

$$V_{Rd,df} = \frac{1.5 \, \tau_0 \, b \, t}{h/b} \sqrt{\left(1 + \frac{\sigma_0}{1.5 \, \tau_0} \right)}$$

■ Shear-sliding

$$V_{Rd,sc} = \frac{1.5 \, \tau_0 + 0.4\sigma_0}{1 + \dfrac{3 \, h \, \tau_0}{b \, \sigma_0 \, \gamma_m}} \frac{b \, t}{\gamma_m}$$

Spandrels

■ Flexure compression

$$M_{Rd,bend} = \frac{2}{3} \, f_{t,eq} \, t \, \frac{h^2}{4}$$

dove: $\quad f_{t,eq} = \dfrac{(\tau_0 + 0.65 \, \sigma_P) \, b_{eff}}{b_h}$

$$V_{Rd,bend} = \frac{2 \cdot M_{Rd,bend}}{l}$$

■ Diagonal cracking

$$V_{Rd,df} = \frac{1.5 \, \tau_0 \, h \, t}{h/l} \sqrt{\left(1 + \frac{\sigma_h}{1.5 \, \tau_0} \right)}$$

■ Shear-sliding

$$V_{Rd,sc} = \tau_0 \, h \, t$$

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Table 21 - *Modal response of the reinforced structure, modeled as an equivalent frame.*

Wall_1 - Masonry Piers													
Pier	b (mm)	t (mm)	h (mm)	N_{sd} (kN)	V_{sd} (kN)	M_{sd} (kNm)	$V_{rd,bend}$ (kN)	$V_{rd,df}$ (kN)	$V_{rd,sc}$ (kN)	V_{rd} (kN)	V_{rd}/V_{sd}	M_{rd} (kNm)	M_{rd}/M_{sd}
10101	2000	350	2880	-409.89	-59.48	-70.16	126.51	95.67	95.90	95.67	0.23	182.17	0.35
				-23.4	48.94	65.26	15.73	36.40	11.20	11.20		22.66	
				-340.73	-102.8	-161.47	127.34	88.05	81.88	81.88		183.37	
				-298.93	-102.8	-86.97	123.48	83.10	73.36	73.36		177.81	
10102	2000	350	2400	-27.81	51.01	73.02	22.30	45.12	13.93	13.93	0.27	26.76	0.37
				-215.76	-78.11	-94.99	127.22	86.71	57.44	57.44		152.66	
				-210.42	-92.91	-104.18	125.34	85.80	56.34	56.34		150.41	
				-171.96	-92.91	-66.49	109.90	79.00	48.31	48.31		131.88	
10103	2000	350	2400	-3.24	20.81	61.9	2.69	36.32	2.47	2.47	0.12	3.23	0.05
				-83.24	-53.1	-57.31	61.54	60.44	28.90	28.90		73.85	
				-76.24	-58.48	-68.37	56.97	58.73	27.24	27.24		68.36	
				-37.78	-58.48	-20.74	29.87	48.24	17.13	17.13		35.85	

Wall_1 - Spandrel												
Spandrel	l (mm)	t (mm)	h (mm)	V_{sd} (kN)	M_{sd} (kNm)	$V_{rd,bend}$ (kN)	$V_{rd,df}$ (kN)	$V_{rd,sc}$ (kN)	V_{rd} (kN)	V_{rd}/V_{sd}	M_{rd} (kNm)	M_{rd}/M_{sd}
10501	1250	350	1250	-124.38	-63.13	5.83	26.25	17.50	5.83	0.05	3.65	0.06
10502	1500	350	1250	108.64	70.68	4.86	26.25	17.50	4.86	0.04	3.65	0.05
				-70.96	-80.37	4.86	26.25	17.50	4.86		3.65	
10503	1250	350	1250	104.6	55.48	5.83	26.25	17.50	5.83	0.06	3.65	0.05
				-72.51	-71.23	5.83	26.25	17.50	5.83		3.65	

5.2.6 DESIGN OF REINFORCEMENT INTERVENTION ON THE STRUCTURE

For reinforcement, the CRM system is used, with components indicated in paragraph 5.2. Verification is carried out for diagonal cracking and flexure compression. Sliding is neglected since the continuity of reinforcement, represented by the presence of adequate anchorage in the foundation, prevents this mechanism. As done for the unreinforced case, the most severe combination is considered for each element. The characteristics of the reinforced masonry are reported in the following table and have been evaluated using the method indicated in paragraph 5.3 (experimental basis 2015) **[09]**:

Double leaf stone masonry		
Compressive strength ($f_{m,R}$)	1,20 f_m	MP_a
Elastic modulus ($E_{m,eq}$)	2617	MP_a
Shear strength ($\tau_{0,R}$)	0,08	MP_a
Volume weight (W_{eq})	21,0	kN/m^3

Out-of-plane mechanism verification

For local checks, what is reported in paragraph 5.1 is taken into consideration, considering the effects of the CRM System on the masonry. This means: improvement of mechanical characteristics, variation of the elastic modulus, variation of the deformation at the first mode. For this last aspect, given the continuity of the reinforcement mesh, it would be overly cautious to consider a simply supported beam scheme; at the same time, considering a beam scheme with double articulation would be a condition difficult to verify mechanically. For this reason, an intermediate behavior is considered, particularly regarding the calculation of the wall's natural period:

- Simply supported beam scheme

$$T_a = 0.64 \, h^2 \sqrt{\frac{A \cdot \gamma_m}{EI \cdot g}}$$

- Fixed beam scheme

$$T_a = 1.79 \, h^2 \sqrt{\frac{A \cdot \gamma_m}{EI \cdot g}}$$

The verification of pier 3, wall 1 is reported. The structural factor is now **q = 3.**

Wall_1 – Masonry pier 3											
Pier	b (mm)	t (mm)	h (mm)	S_a (support)	S_a (interlock)	F_a support	F_a interlock	p medium (kN/m)	M_{sd} medium (kN/m)	M_{Rd} (kNm)	M_{Rd}/M_{Sd}
10301	3000	350	2560	0.32	0.42	47.11	61.65	42.48	25.13	88.54	3.52
10302	3000	350	2000	0.41	0.48	35.43	41.63	38.52	14.76	76.99	5.22
10303	3000	350	2000	0.53	0.61	19.78	23.08	21.43	8.24	65.88	8.00

The elements have been verified.

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In-plane mechanism verification

For the checks, the following relationships are adopted

Masonry piers

- Flexure compression

$$M_{Rd,bend} = \frac{\sigma_0\, b^2\, t}{2}\left(1 - \frac{\sigma_0}{0.85\, f_m}\right)$$

$$V_{Rd,bend} = \frac{2 \cdot M_{Rd,bend}}{h}$$

- Diagonal cracking

$$V_{Rd,df} = \frac{1.5\, \tau_0\, b\, t}{h/b}\sqrt{\left(1 + \frac{\sigma_0}{1.5\, \tau_0}\right)}$$

Spandrels

- Flexure compression

$$M_{Rd,bend} = \frac{2}{3}\, f_{t,eq}\, t\, \frac{h^2}{4}$$

dove: $\quad f_{t,eq} = \dfrac{(\tau_0 + 0.65\, \sigma_P\,)\, b_{eff}}{b_h}$

$$V_{Rd,bend} = \frac{2 \cdot M_{Rd,bend}}{l}$$

- Diagonal cracking

$$V_{Rd,df} = \frac{1.5\, \tau_0\, h\, t}{h/l}\sqrt{\left(1 + \frac{\sigma_h}{1.5\, \tau_0}\right)}$$

Table 22 - *Strength checks for masonry reinforced with CRM system*

Pier	b (mm)	t (mm)	h (mm)	N_{sd} (kN)	V_{sd} (kN)	M_{sd} (kNm)	$V_{rd,bend}$ (kN)	$V_{rd,dc}$ (kN)	V_{rd} (kN)	V_{rd}/V_{sd}	M_{rd} (kNm)	M_{rd}/M_{sd}
							Wall_1 - Masonry piers					
10101	2000	350	2880	-86.24	24.57	48.1	63.32	195.47	63.32	2.32	91.18	1.90
				-310.78	-59.62	-90.7	164.07	260.91	164.07		236.27	
				-263	-59.62	-41.67	150.39	248.43	150.39		216.57	
				-171.93	48.54	85.55	112.70	222.72	112.70		162.29	
10102	2000	350	2400	-70.95	21.52	49.33	63.71	228.23	63.71	2.23	76.45	1.22
				-200.05	-50.09	-60.05	151.16	277.16	151.16		181.39	
				-156.1	-50.09	-11.91	125.52	261.53	125.52		150.62	
				-74.13	29.72	65.26	66.31	229.56	66.31		79.57	
				-196.87	-58.29	-65.27	149.45	276.06	149.45		179.34	
10103	2000	350	2400	-42.89	3.76	45.37	39.84	216.14	39.84	1.01	47.81	1.05
				-82.53	-38.09	-41.43	73.06	233.04	73.06		87.67	
				-78.97	-40.9	-47.25	70.21	231.57	70.21		84.26	
				-44.51	-40.9	-0.98	41.27	216.85	41.27		49.52	

Spandrel	l (mm)	t (mm)	h (mm)	V_{sd} (kN)	M_{sd} (kNm)	$V_{rd,bend}$ (kN)	$V_{rd,dc}$ (kN)	V_{rd} (kN)	V_{rd}/V_{sd}	M_{rd} (kNm)	M_{rd}/M_{sd}
						Wall_1 - Spandrels					
10501	1250	350	1250	-77.57	-35.86	82.96	223.26	82.96	1.07	3.65	1.45
10502	1500	350	1250	68.57	37.74	69.14	267.91	69.14	1.01	3.65	1.13
				-30.6	-45.86	69.14	267.91	69.14		3.65	
10503	1250	350	1250	65.65	30.53	82.96	223.26	82.96	1.26	3.65	1.31
				-32.12	-39.61	82.96	223.26	82.96		3.65	

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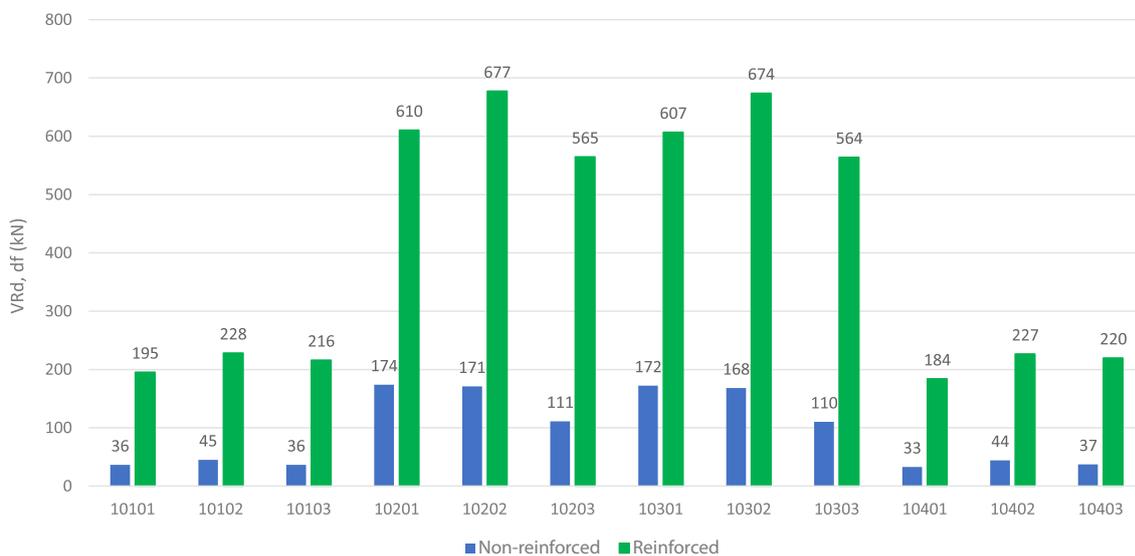
5.2.7 CONCLUSIONS

In this chapter, the results of a linear dynamic analysis on a masonry building modeled using an Equivalent Frame Model have been presented, both in its existing state and in its state reinforced by the CRM system. As evident from the summary tables below (highlighting the resistance capacities for various collapse mechanisms of masonry piers and spandrels, comparing the unreinforced and reinforced elements), the introduction of the CRM system for masonry consolidation has led to a significant increase in elements meeting the requirements for seismic actions.

Whereas the failures of unreinforced elements are more related to the tensile strength of the masonry (diagonal cracking mechanism for masonry piers and spandrels, flexure for spandrels), the reinforcing contribution of the CRM system enables a substantially higher number of structural elements to meet the requirements. Based on the calculation and modeling assumptions of the structure, including verifications both out-of-plane and in-plane for the elements composing the masonry walls, the use of the CRM system allows for a complete seismic upgrade of the building.
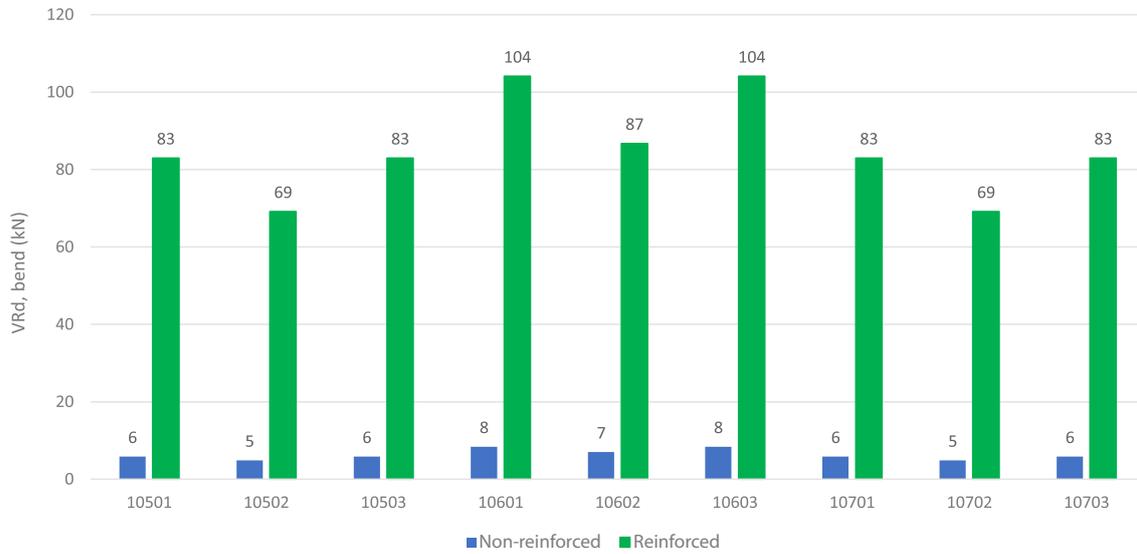
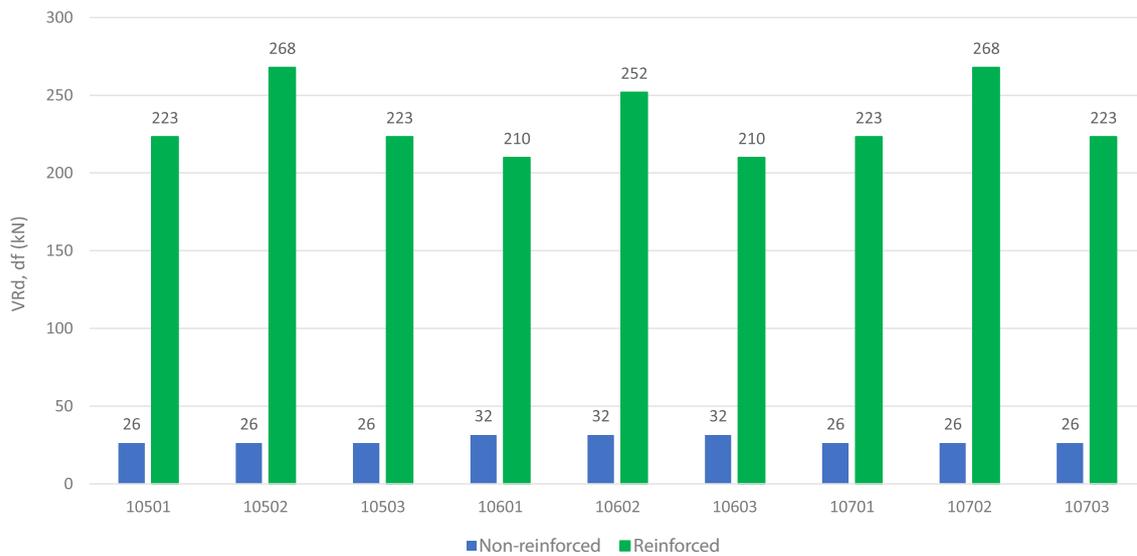Resistance by flexural compression - NR/R comparison - Piers wall 1

Element	Non-reinforced	Reinforced
10101	16	63
10102	22	64
10103	3	40
10201	249	357
10202	187	288
10203	35	70
10301	245	352
10302	179	282
10303	33	68
10401	8	41
10402	20	60
10403	4	47

Resistance by diagonal cracking - NR/R comparison - Piers wall 1

Element	Non-reinforced	Reinforced
10101	36	195
10102	45	228
10103	36	216
10201	174	610
10202	171	677
10203	111	565
10301	172	607
10302	168	674
10303	110	564
10401	33	184
10402	44	227
10403	37	220
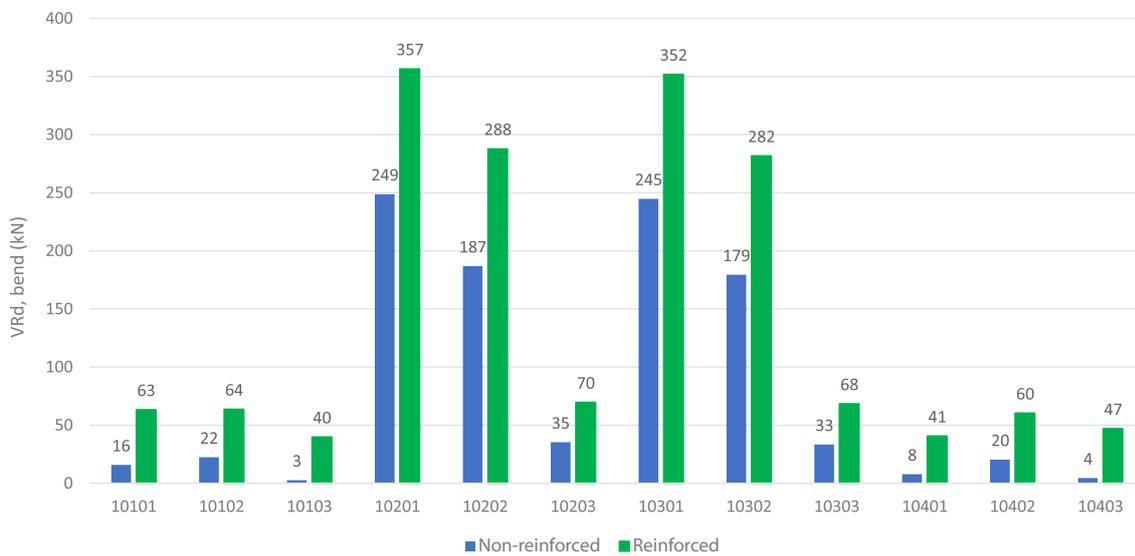
Resistance by pressure deflection - NR/R comparison - wall 1 spandrels

VRd, bend (kN)

	Non-reinforced	Reinforced
10501	6	83
10502	5	69
10503	6	83
10601	8	104
10602	7	87
10603	8	104
10701	6	83
10702	5	69
10703	6	83

Resistance by diagonal cracking - NR/R comparison - wall 1 spandrels

VRd, df (kN)

	Non-reinforced	Reinforced
10501	26	223
10502	26	268
10503	26	223
10601	32	210
10602	32	252
10603	32	210
10701	26	223
10702	26	268
10703	26	223

In cases where instead of evaluating individual structural elements through linear analysis, one wishes to undertake nonlinear analysis on the building, and thus fully exploit the nonlinear behavior provided by the CRM system RI-STRUTTURA in its various combinations (mortar, mesh, angles, and connectors), the global behavior analyses of the building in its two configurations (existing state and design state) would show even more significant differences. These differences can be appreciated by examining the following graph (see next page), which depicts the behavior of the building described in this chapter undergoing a nonlinear static analysis (push over analysis) in the X direction, both pre and post consolidation, under the assumption of design according to the new Fibre Net 2023 approach.
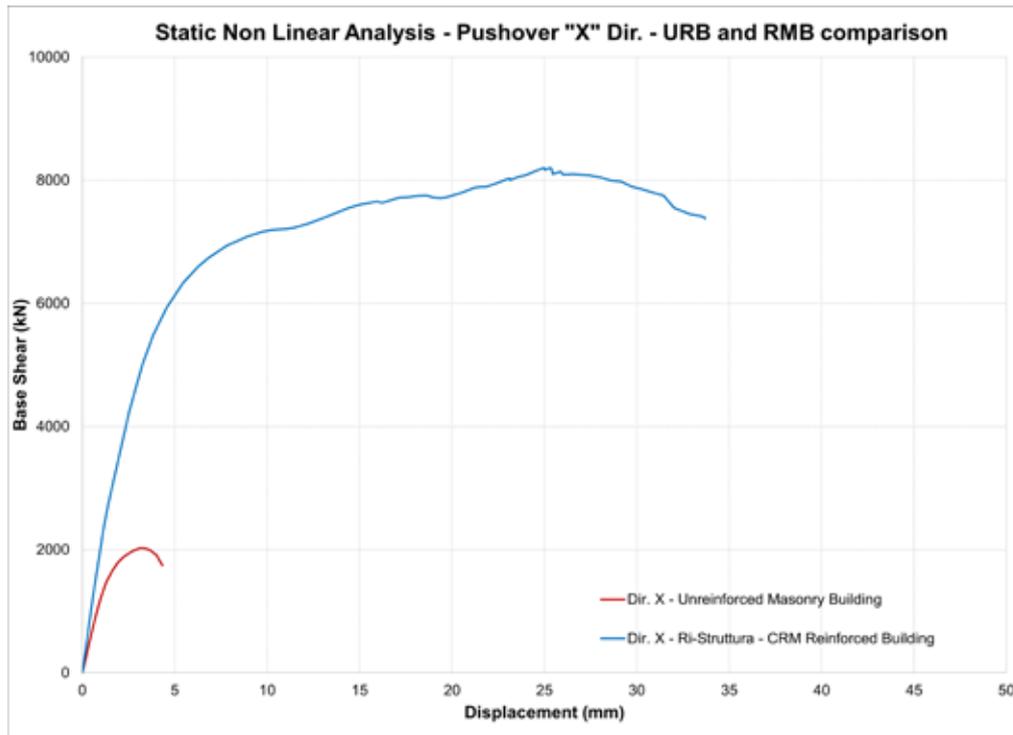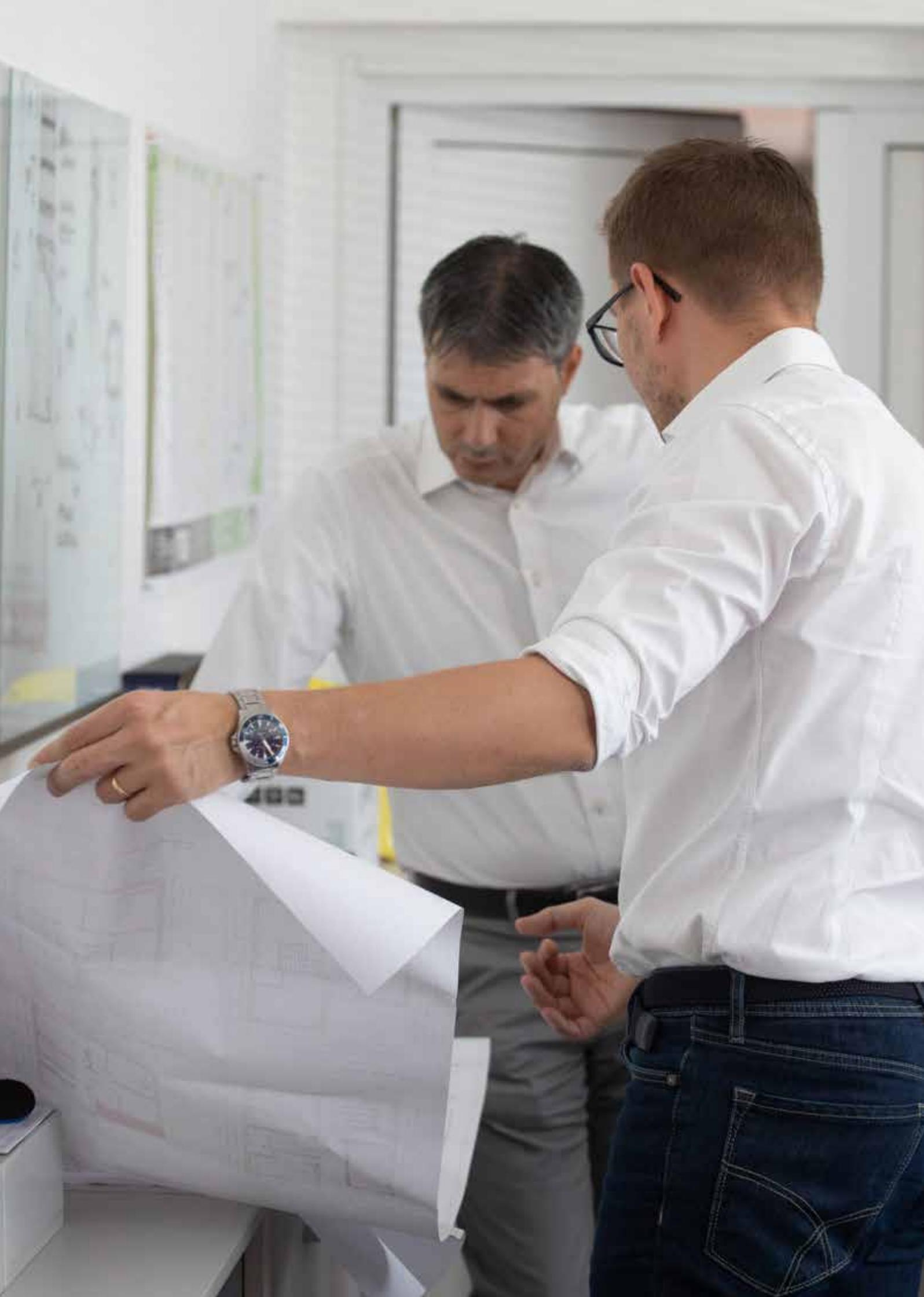
Evaluation of the seismic risk class

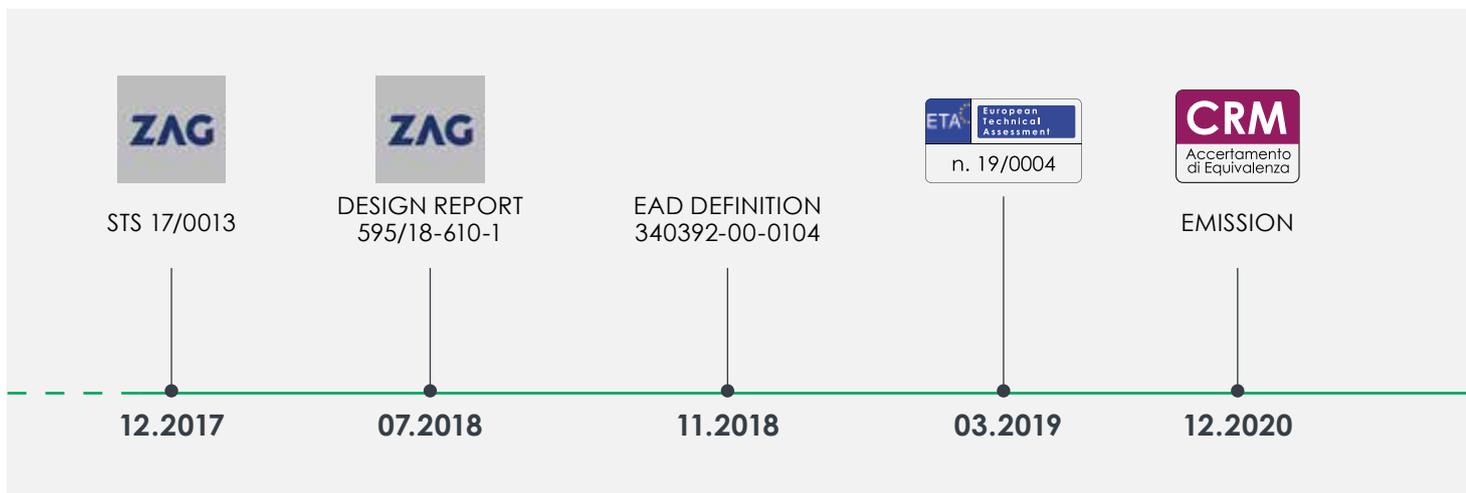**PRE-INTERVENTION**

POST-INTERVENTION

6

THE QUALIFICATION PROCESS OF THE CRM SYSTEM

Since 2009, the company has embarked on the path of Material Qualification, despite the absence of existing documents and standards regulating it. The extensive volume of experimental results naturally led the company to pursue the ambitious project of defining ex novo, filling an evident regulatory gap and in collaboration with Certification Bodies, the Qualification and Design rules for the use of the company's Systems. During the qualification process, it was necessary to identify the System and give it a name that ensured its uniqueness: parallel to the known Systems such as **FRP** (Fiber Reinforced Polymer) and **FRCM** (Fiber Reinforced Cementitious Matrix), the acronym **CRM** (Composite Reinforced Mortar) devised by us during the obtaining of the Slovenian Technical Approval, allowed us to uniquely define a new consolidation system, different from others in terms of components and mechanical behavior: so much experimentation was carried out on this System and the results and certifications obtained by the Company were so significant that the term CRM has now become a common term in the construction industry to identify a very specific intervention strategy.

Considering, therefore, the need to qualify the System to ensure, in addition to experimental credibility, the formal one for normal use in the construction sector, Fibre Net turned to **ZAG** - Slovenian National Building And Civil Engineering Institute (Zavod za gradbeništvo Slovenije) in 2017 to obtain the Slovenian National Technical Approval **STS**: within this document, the main results of the experimental campaigns in collaboration with the Universities of Trieste, Perugia, and Salento are recognized and validated, translated into material

and components qualification rules of the CRM System and, extremely importantly, into design rules of the System applied to masonry. The dialogue and presentation of the results obtained to the Technical Staff of ZAG allowed us to obtain, in addition to the Technical Approval, also an exclusive System Design Guideline, recognized internationally.

Armed with this obtained Certification, we turned to the Central Technical Services of the Ministry of Infrastructure and Sustainable Mobility to assert the principle of Equivalence of the Qualification obtained in another member state of the European Community. With the introduction of the Qualification Guideline for CRM Systems at the Italian level, the close dialogue with the STC Technical Staff allowed us to achieve, in December 2020, a very important result, namely obtaining the Equivalence Certificate to another technical specification, pursuant to point 11.1 of the D.M.17.1.2018, formalizing the structural use of CRM System components in the Italian territory. In parallel, Fibre Net turned to ITC CNR, Italian **TAB** (Technical Assessment Body) for **EOTA** (European Organization for Technical Assessment) to initiate the **ETA** (European Technical Assessment) procedure with the creation of the **EAD** (European Assessment Document) **[32]** and to ensure structural use throughout the European territory. The experience and daily interaction with Sector Institutions allowed us to obtain the European Technical Approval very quickly and, finally, gave us the opportunity to **CE** mark the System components in September 2021, a worthy recognition for a twenty-year innovation path.

STS 17/0013	DESIGN REPORT 595/18-610-1	EAD DEFINITION 340392-00-0104	n. 19/0004	EMISSION
12.2017	**07.2018**	**11.2018**	**03.2019**	**12.2020**

7

REGULATORY ASPECTS

The reference regulations for the design and use of the CRM (Composite, Reinforced Mortar) reinforcement system are the Technical Standards for Construction known as NTC 2018 and the related explanatory circulars as per the following decrees:

▪ Ministerial Decree of January 17, 2018 - Technical Standards for Construction;
▪ Circular of January 21, 2019, no. 7 of the C.S.LL.PP - Instructions for the application of the update of the "Technical Standards for Construction" referred to in the Ministerial Decree of January 17, 2018.

As known, under Chapter 11 of the NTC 2018, all materials for structural use must be identified, qualified, and accepted by the construction manager. The methods for their identification and qualification are established by the same NTC, distinguishing cases A-B-C; for further details, please refer to NTC2018.
Specifically, the CRM reinforcement technique falls under case C "Materials and products for structural use for which the manufacturer achieves identification and qualification with CE marking through ETA or through a CVT Technical Evaluation Certificate." The CVT will be issued by the Superior Council of Public Works following an appraisal and based on guidelines issued by the same council of public works through the STC. The procedures for the identification, qualification, and control of the CRM System at the national level are described in:
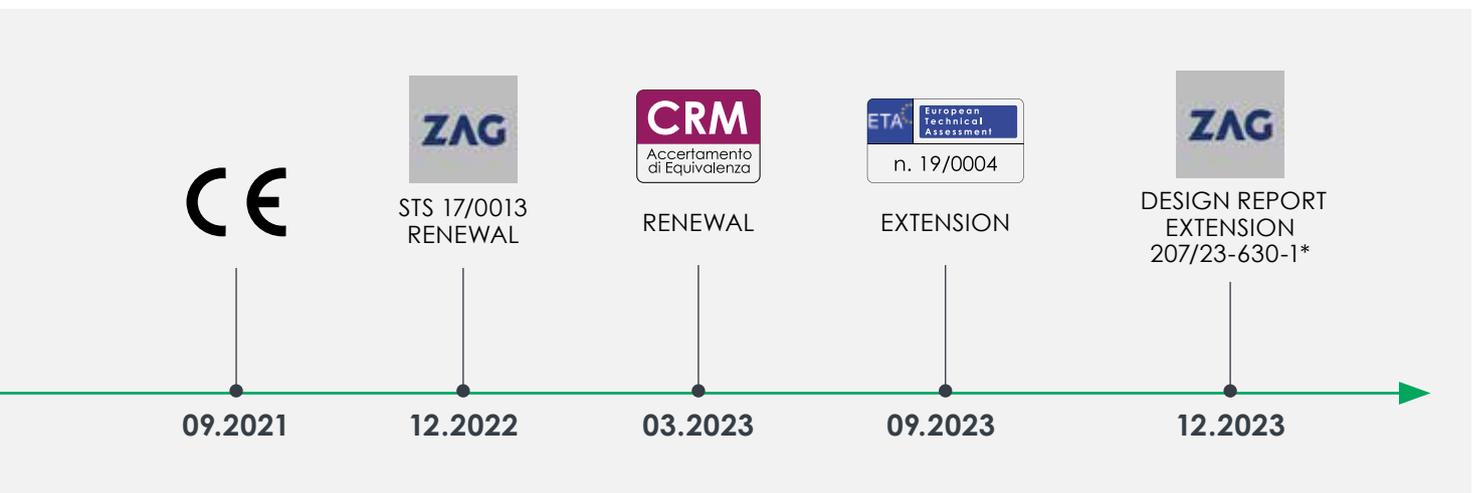
"Guidelines for the identification, qualification, and acceptance control of preformed mesh systems in fiber-reinforced composite materials with a polymeric matrix to be used for the structural consolidation of existing buildings with the CRM reinforced plaster technique (Composite Reinforced Mortar)" issued by the C.S.LL.PP in May 2019, which constitutes a document of proven validity under Chapter 12 of the NTC 2018.

The design of the CRM reinforcement system according to the aforementioned regulations can be carried out with two substantially different methods:
▪ tabular
▪ analytical-experimental

The simplified tabular method involves applying corrective/improvement coefficients to the mechanical parameters of the existing state masonry. These coefficients are contained and described in Table C8.5.II of the Circular of January 21, 2019. The analytical method allows using specific formulas to dimension the various components of the reinforcement system (paragraph C8.5.3.1 MASONRY CONSTRUCTIONS). The major novelty of the RI-STRUTTURA System lies precisely in the possibility of "ad hoc" dimension the reinforcement system according to specific needs. For further details, please refer to Chapter 5.

*Processing results CONSTRAIN PROJECT

CE	ZAG STS 17/0013 RENEWAL	CRM Accertamento di Equivalenza RENEWAL	ETA European Technical Assessment n. 19/0004 EXTENSION	ZAG DESIGN REPORT EXTENSION 207/23-630-1*
09.2021	**12.2022**	**03.2023**	**09.2023**	**12.2023**

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8
ACCEPTANCE OF MATERIAL ON SITE

Regarding the acceptance of materials, the "Guidelines for the identification, qualification, and acceptance control of CRM systems (Composite Reinforced Mortar)" specify in Chapter 9 that controls are mandatory, must be carried out under the responsibility of the Works Manager, must be conducted for each shipment batch with reference to the production batch, and must involve all components of the CRM System supplied. Acceptance consists of verifying that the products of each shipment batch are covered by a valid Technical Evaluation Certificate or, alternatively, provided with CE marking, by verifying the Declaration of Performance (DoP) in relation to European legislation on construction products.

Extract from the **"Guidelines for the Identification, Qualification, and Acceptance Control of Prefabricated Mesh Systems in Composite Fiber-Reinforced Polymer Matrix Materials for Structural Consolidation of Existing Buildings Using the CRM (Composite Reinforced Mortar) Plaster Technique."**

CHAPTER 9: ON-SITE ACCEPTANCE PROCEDURES

The acceptance checks on-site:

- are mandatory and must be carried out under the supervision and responsibility of the Construction Manager;
- must be sampled within each shipment batch with reference to the production batch and must involve all components of the CRM System being supplied.

Three specimens per component of the reinforcement systems to be installed are required, taking into account the possible different nature of the phases (especially the reinforcement unit weight) and any different characteristics of meshes in both directions. The dimensions should be as indicated for the tensile test (Annex 1).
The tests to be performed are exclusively those of traction on the FRP components of the system, as described in Annex 1. Furthermore, tests on the mortars to be used are prescribed, based on two specimens for each shipment batch, to verify the characteristics declared by the Manufacturer and referred to in the Installation Manual. For this purpose, the same reference standards used for the qualification of raw materials referred to in point 4.2 are adopted.
These specimens must be sent by the Construction Manager to a Laboratory as per article 59 of DPR 380/2001, and the required tests must be carried out on them. For each specimen, the values of the tensile breaking strength and the elastic modulus must not be lower than the corresponding nominal values declared in the Product Data Sheet.
The properties of the mortar must meet the values declared by the Manufacturer. The request for tests at the Laboratory must be signed by the Construction Manager and must contain indications regarding the samples of mesh and bonding agent taken. If the Construction Manager fails to sign the request for tests, the certifications issued by the Laboratory cannot be valid for the purposes of this document, and this must be explicitly stated on the certificate itself.
If the aforementioned checks fail to meet the requirements, even for just one of the measured parameters, they must be repeated by sampling and testing 3 additional specimerns of the FRP component(s) of the CRM System from the lot under examination, or 2 additional samples of mortar from the same shipment batch. If, for all the samples, the values of the mechanical characteristics to be examined are greater than or equal to the corresponding acceptance values, the delivered batch can be considered compliant. If, for just one of the specimens, the value of one of the two mechanical characteristics is lower than the corresponding acceptance value, both the specimen and the test method must be carefully analyzed.
If a defect is present in the sample or there is reason to believe that an error occurred during the test, the result of the test itself must be disregarded. In this case, it is necessary to take an additional (single) specimen and verify compliance with the acceptance requirements.
In all other cases, the negative test result must be communicated to the STC. The entire shipment batch is considered non-compliant and as such must not be used for the intended structural reinforcement.
The certificates issued by the Laboratories must necessarily contain at least the data indicated in § 7.1.

In addition to carrying out acceptance checks, the Construction Manager, during the acceptance phase, must verify that the products constituting each shipment batch are covered by a valid Technical Evaluation Certificate, of which a copy must be attached to the shipping documents.

In the case of materials and products bearing the CE Marking, it is the responsibility of the Construction Manager, during acceptance, to ensure the possession of the Marking itself and to request from each Manufacturer, for each different product, the Certificate of Conformity to the harmonized part of the specific European standard, or the Declaration of Performance (DOP) in relation to the applicable European legislation on construction products. In any case, it is also the responsibility of the Construction Manager to verify that the products delivered to the construction site fall within the types specified in the aforementi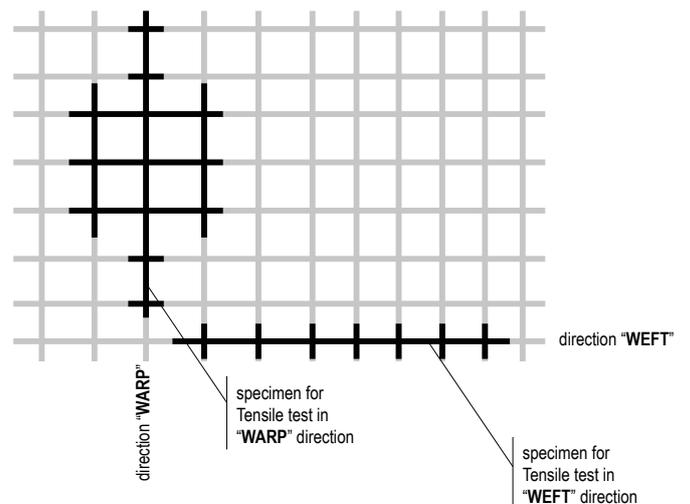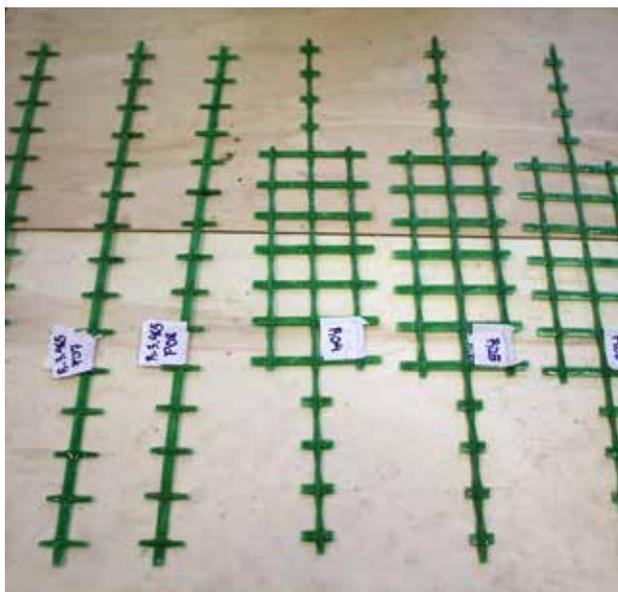oned documentation. The Construction Manager, before the installation, is required to reject any non-compliant supplies, with the responsibilities of the Manufacturer / Distributor remaining unchanged. For traceability purposes, if necessary, the Construction Manager must carefully annotate the location, within the framework of the consolidated structure, of the reinforcement systems corresponding to the various shipment batches, transmitting the annotations, duly signed, to the Contractor or the intervention executor. The Manufacturer must ensure proper archiving of the accompanying documentation for the materials, guaranteeing its availability for at least ten years.

For product traceability purposes, the Contractor must also ensure the preservation of the same documentation, along with markings or recognition labels, and any annotations transmitted by the Construction Manager, until the completion of static assessment operations.

SAMPLING PROCEDURE FOR RI-STRUTTURA SYSTEM COMPONENTS

For each component in GFRP, at least three samples must be taken; the test to be performed is the tensile test to determine the strength and modulus of elasticity. Below is described the sampling method for each component of the system.

GFRP mesh

It is recommended to take a GFRP mesh panel measuring 100 x 100 cm² from which the laboratory will then obtain three + three specimens (three for each direction in case the mesh is unbalanced). Depending on the nature of the mesh, particularly the weft and warp node, the dimensions of the specimens to be subjected to laboratory testing may vary. In fact, if the weft-warp node is made by twisting one of the two threads, the sampling can be carried out according to the methods shown in the following image.

direction "WARP"

direction "WEFT"

specimen for Tensile test in "WARP" direction

specimen for Tensile test in "WEFT" direction

It is recommended to include in the sampling report references to the specific Delivery Note (DDT) and its corresponding batch. The mesh panel must be identified using indelible labeling with the respective identification symbol by the Construction Manager.

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Angle reinforcement in GFRP

The test must be conducted on a single bar that forms the preformed angle in the standard length.
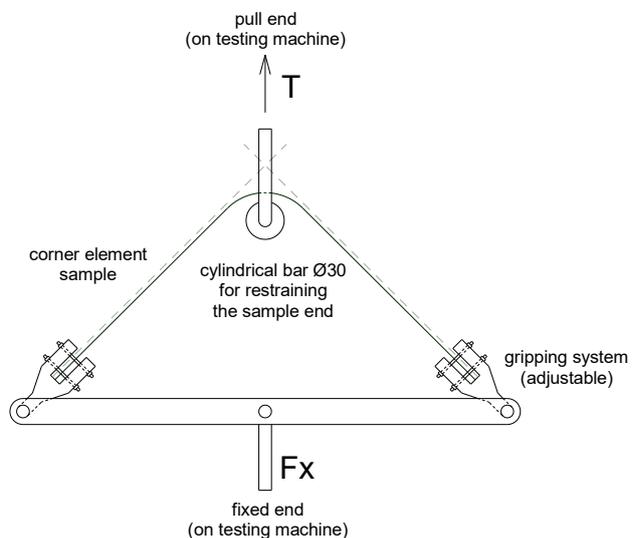
Connector in GFRP

The test must be conducted on elements with a length defined according to what is reported in the document CNR DT 203/2006, summarized below:

The length l_p must meet the following requirements:

$$l_p \geq 100 + 2 \cdot l_a \qquad [lengths\ in\ mm],$$

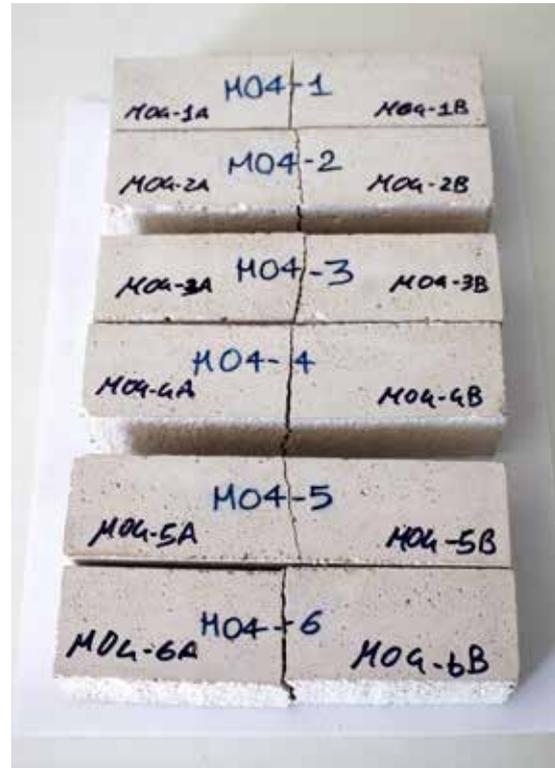$$l_p \geq 40 \cdot d_b + 2 \cdot l_a$$

where:

l_a grip length

d_b maximum connector size

l_c Minimum distance beyond which the extensometer must be positioned

$$l_c \geq l_a + 8d_b$$

Cement - or lime-based mortar

For each batch of mortar shipment, it is necessary to take 2 specimens (two prisms with dimensions of 40x40x160 mm).

INTERPRETATION OF RESULTS

The acceptance tests for GFRP components of the CRM System are considered satisfactory if, for each specimen, the values of tensile strength and modulus of elasticity are equal to or greater than the nominal values declared in the Product data sheet. Regarding the mortar, the results must meet the values declared by the Manufacturer in accordance with the specifications according to UNI EN 998 parts 1 and 2 and/or UNI EN 1504 parts 2 and 3.

In summary, for all samples, if the values of the mechanical characteristics to be examined are greater than or equal to the corresponding acceptance values, the delivered batch can be considered compliant.
In the event that the acceptance tests on the GFRP components are not satisfied, even for only one of the measured quantities, the tests must be repeated on additional 3 samples from the batch in question. The same procedure applies to the mortar: additional 2 samples of mortar from the same shipment batch must be taken. If the value of one of the two mechanical characteristics is lower than the corresponding acceptance value, a careful examination of the specimen and the test method is essential: if defects are found or an error occurred during the test, the obtained result must be disregarded, and an additional sample must be taken.

RI-STRUTTURA CRM System Component	Image of the component	Type of test	Number of samples	Verification
GFRP mesh		tensile	3 specimens for each direction	*Average values f_u e E_f> Class values or value declared by the manufacturer in case of CE marking*
Angle reinforcement in GFRP		tensile	3 specimens obtained in the direction of the angle	*Average values f_u e E_f> Class values or value declared by the manufacturer in case of CE marking*
Connector in GFRP		tensile	3 specimens	*Average values f_u e E_f> Class values or value declared by the manufacturer in case of CE marking*
Mortar		flexure /compression strength	2 prisms 40x40x160 mm	> Declared values

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FAC-SIMILE
SAMPLING REPORT

Order _____

Client _____

Contractor _____

Presents _____

Place _____

Report of withdrawal of CRM Reinforcement System components

Manufacturer: FIBRE NET SPA

Product qualified with CE marking: 0970-CPR-0154/CE/FPC21

Qualification certificate:

DoP: FB 01 DDP 03IT01

DDT: _____ **Batch**: _____

The undersigned, in accordance with Chapter 11 of the NTC 2018, orders the sampling of the following system components:

- Mesh portion with dimensions of 1x1 m^2
- 3 connectors of the FB-Con type
- 3 Angle reinforcement
- 2 mortar prisms 40x40x160

Sample labels: the samples taken were identified by a label and tag applied with indelible marker. Subsequently, the samples were photographed.

The mesh was labeled with the code _____

The FB-Con connectors with the codes. _____

3 Angle reinforcement with the codes. _____

The mortar prisms with the codes. _____

Request: Tensile tests to determine tensile strength and deformation properties until failure, as well as elastic modulus. The mortar prisms will undergo a flexural-compression test according to the LG CRM version 2019.

Laboratory: the tests must be carried out at a laboratory authorized under article 59 of Legislative Decree 380/2001.

Delivery: the samples are handed over to the company, which will keep them in custody and will arrange for their transfer together with this report to the laboratory that will perform the above-mentioned tests.

_____ date _____   The Structural Works Director

                                                    _____

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Palace "Lucentini-Bonanni",
L'Aquila

9
TYPICAL DRAWINGS

MASONRY

LIST OF DRAWINGS

REINFORCEMENT ON TWO SIDES

▶	**CRM 00.a**	GENERAL FRAMING
▶	**CRM 01.a**	CONNECTIONS OF L-SHAPE AND T-SHAPE MASONRY WALLS
▶	**CRM 02.a-b-c**	INSTALLATION PHASES IN THE PRESENCE OF OPENINGS (doors/french doors)
▶	**CRM 03.a-b-c**	INSTALLATION PHASES IN THE PRESENCE OF WINDOWS
▶	**CRM 04.a**	CONNECTIONS - Foundation connection
▶	**CRM 04.b**	CONNESSIONI - Collegamento di piano
▶	**CRM 04.c**	CONNECTIONS - Floor connection with balcony
▶	**CRM 04.d**	CONNECTIONS - Roof connection with kerb
▶	**CRM 04.e**	CONNECTIONS - Roof connection without kerb

REINFORCEMENT ONE SIDE

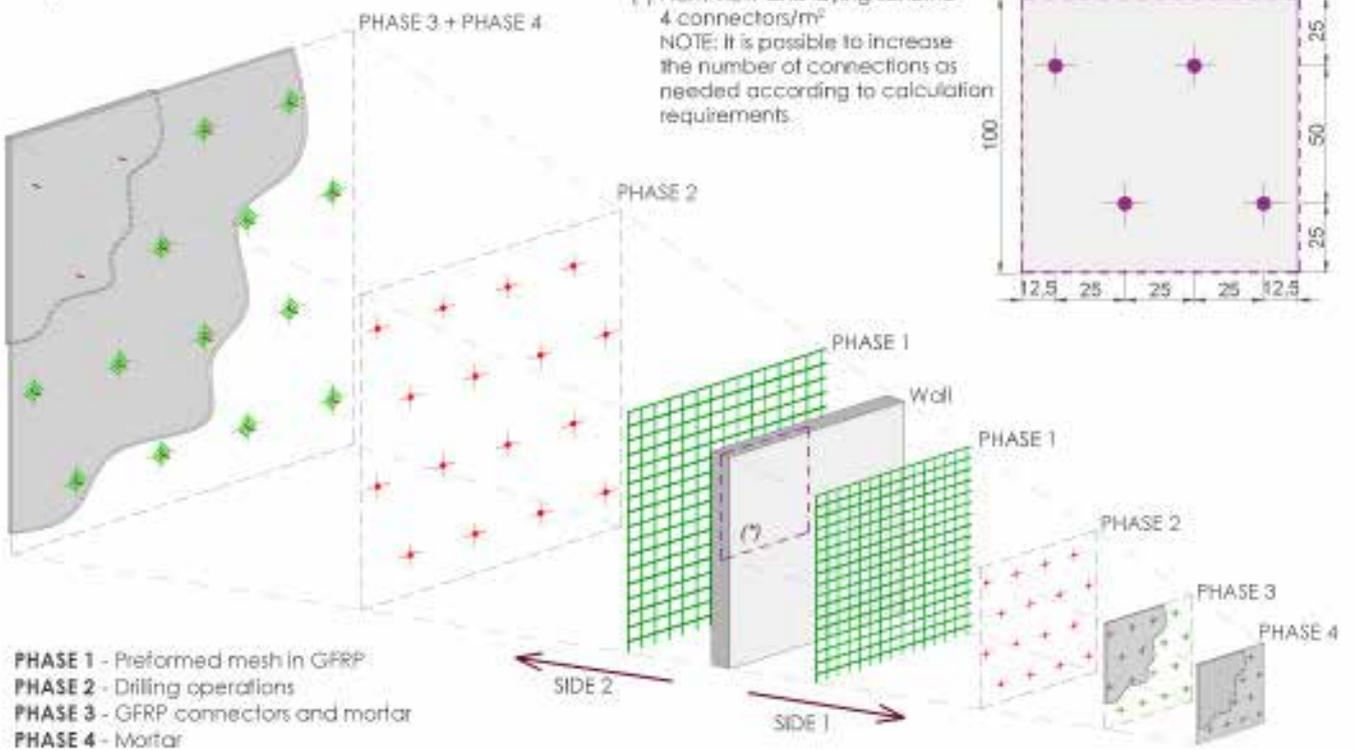|		
▶	**CRM 05.a**	GENERAL FRAMING
▶	**CRM 06.a**	CONNECTIONS OF L-SHAPE AND T-SHAPE MASONRY WALLS
▶	**CRM 07.a-b-c**	INSTALLATION PHASES IN THE PRESENCE OF OPENINGS (doors/french doors)
▶	**CRM 08.a-b-c**	INSTALLATION PHASES IN THE PRESENCE OF WINDOWS
▶	**CRM 09.a**	CONNECTIONS - Foundation connection
▶	**CRM 09.b**	CONNECTIONS - Floor connection
▶	**CRM 09.c**	CONNECTIONS - Floor connection with balcony
▶	**CRM 09.d**	CONNECTIONS - Roof connection with kerb
▶	**CRM 09.e**	CONNECTIONS - Roof connection without kerb

⬇ **DOWNLOAD AREA**

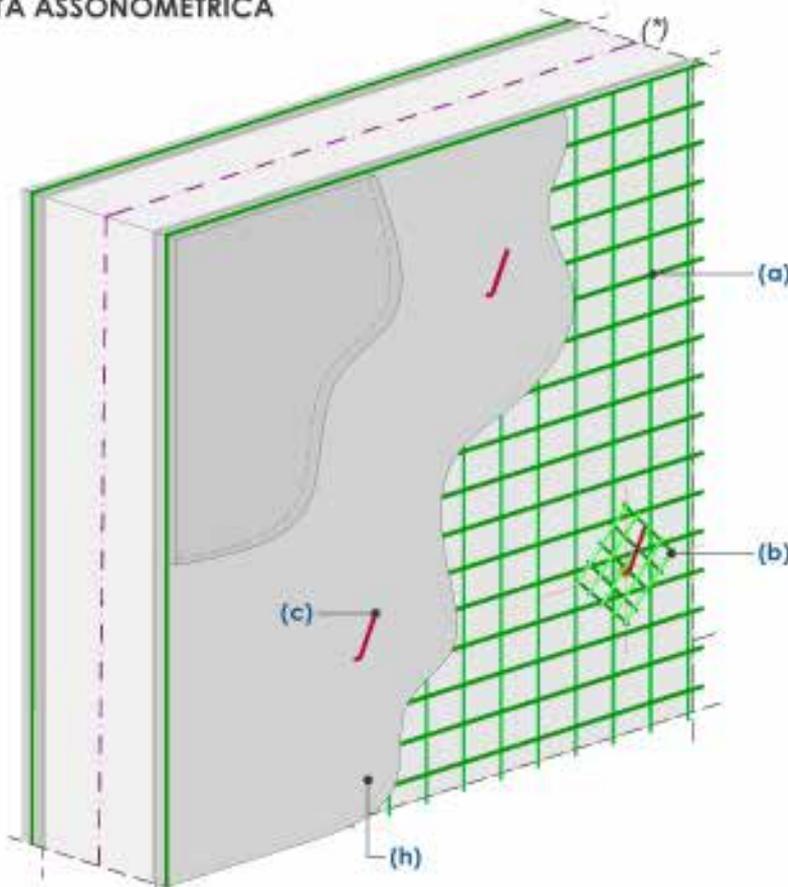**To download the complete package of standard drawings, please contact our offices.**

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WORKING PHASES - CRM REINFORCEMENT ON TWO SIDES
Perspective view

(*) Front view and laying scheme
4 connectors/m²
NOTE: It is possible to increase
the number of connections as
needed according to calculation
requirements.

100

25
100
50
25

12.5   25   25   25   12.5

PHASE 3 + PHASE 4

PHASE 2

PHASE 1

Wall

PHASE 1

(*)

PHASE 2

PHASE 3

PHASE 4

SIDE 2

SIDE 1

PHASE 1 - Preformed mesh in GFRP
PHASE 2 - Drilling operations
PHASE 3 - GFRP connectors and mortar
PHASE 4 - Mortar

VISTA ASSONOMETRICA

(*)

(a)

(b)

(c)

(h)

SECTION DETAIL
scale 1:10

(b)

(a)

(h)

(c)

(d)

t_{wall}
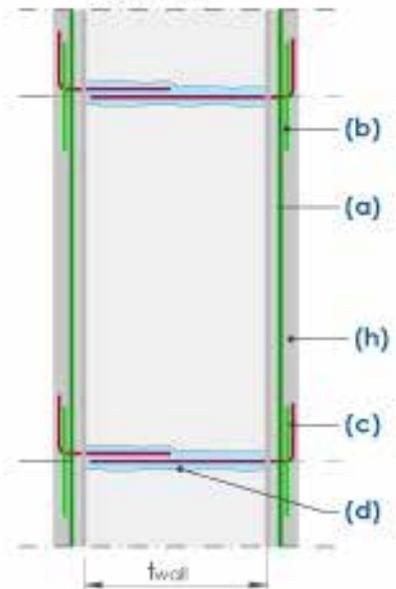
Legend:

Wall	
Undercoat	
GFRP mesh	
"L" shaped connector	
Distribution pad	
Mortar	

REINFORCED PLASTER - CRM SYSTEM IN G.F.R.P.
MASONRY CONSOLIDATION ON TWO SIDES
GENERAL FRAMEWORK

$C \in$

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WORKING PHASES FOR 1 M² OF WALL REINFORCED ON TWO SIDES WITH CRM SYSTEM
Isometric view

PHASE 1

PHASE 2
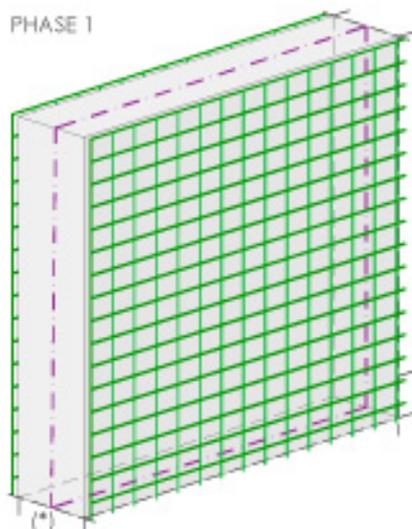
PHASE 3 - PHASE 4

Laying of preformed GFRP mesh for the hole marking and shaping in correspondence with the openings.
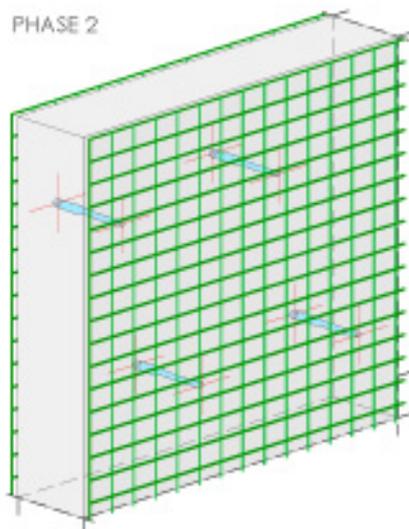
Execution of through holes Ø12 for single connector and Ø18 for areas of overlap with compressed air cleaning.
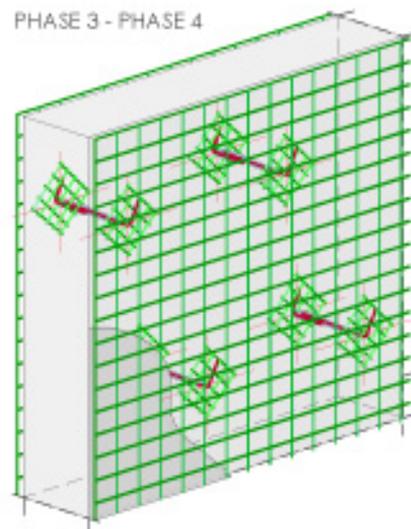
Filling of the holes, laying of the connectors and the GFRP distribution pads. Application of mortar.

DESCRIPTION OF THE WORKS

The intervention involves preparing the support by removing existing layers and subsequently reconstructing the damaged parts of the wall, completing the preparation with the application of the undercoat.

Subsequently, the works for the reinforcement implementation are articulated in the following phases:

1. placing **FBMESH** net and **FBANG** corner elements on the middle plane of the plaster with a 15/20 cm overlap at joints and in corners. Marking on the wall the position of the holes and shaping it accordingly to the openings;

2. drilling of the holes using a rotary drill;

3. Injection of the previously drilled holes with two-component chemical anchor **INTEGRA FIXA VINYL15** and laying of the distribution pad **FBFAZ33X33** with **FBCONL** connector;

4. Proceed with the application of **MATERIA/EPOCA** mortar plaster with a thickness of 3 cm, after wetting the surface to refusal without causing water stagnation.

NOTE: Depending on the thermo-hygrometric conditions, it is recommended to wet the wall the day before plaster application.

IDENTIFICATION OF CRM MATERIALS

(a)   **FBMESH** (Preformed mesh in GFRP)

(b)   **FBFAZ33X33T96AR** (Preformed mesh distribution pad in GFRP)

(c)   **FBCONL** (Preformed "L" connector in GFRP)

(d)   **INTEGRA FIXA VINYL 15** (Two-component chemical anchor)

(e)   **FBANG** (Preformed right-angle element in GFRP)

(f)   **FB-TUP10-VAR1A/2A** (Bar in fiber-reinforced composite material)

(g)   **FB-HBAR10** (Austenitic stainless steel helical bar)

(h)   **MALTA MATERIA/EPOCA** (Pre-mixed eco-friendly lime-based compound)

Terrazza Vasariana,
Florence

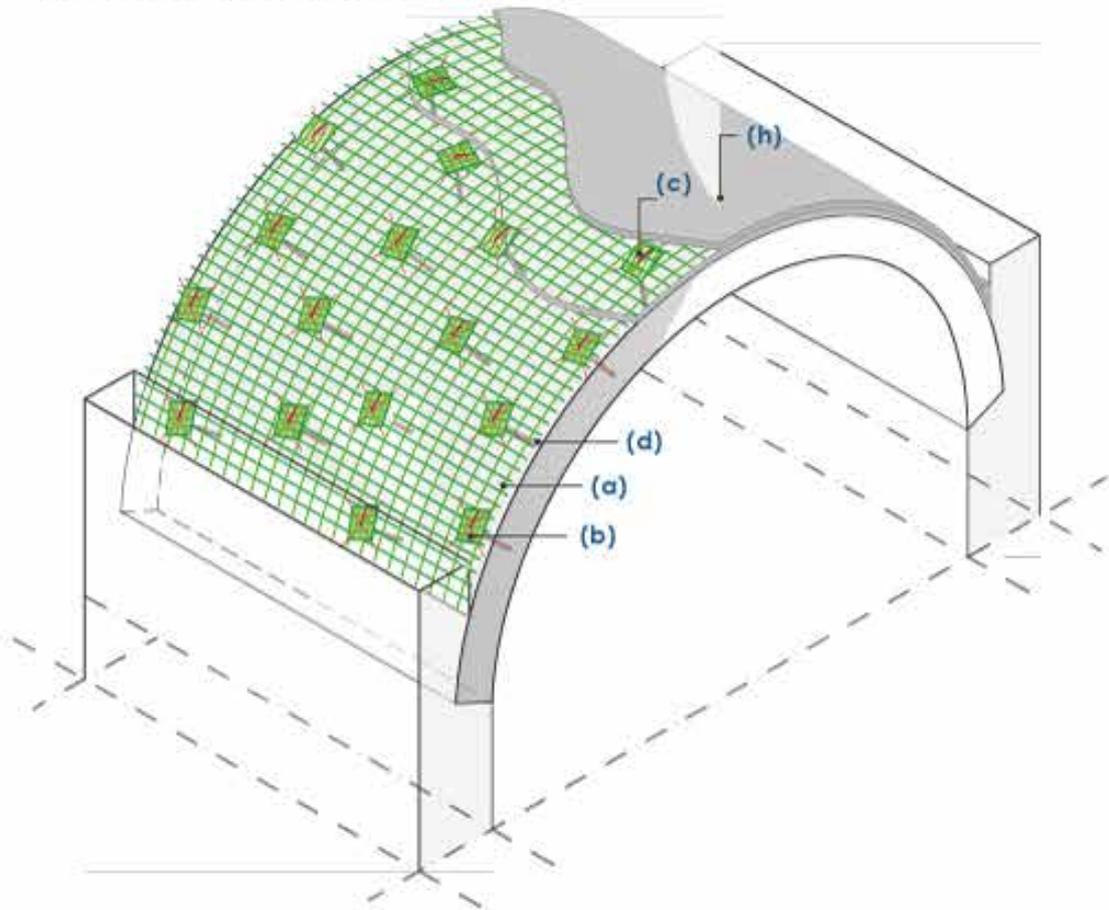# BARREL VAULTS, ARCHES, AND PILASTERS
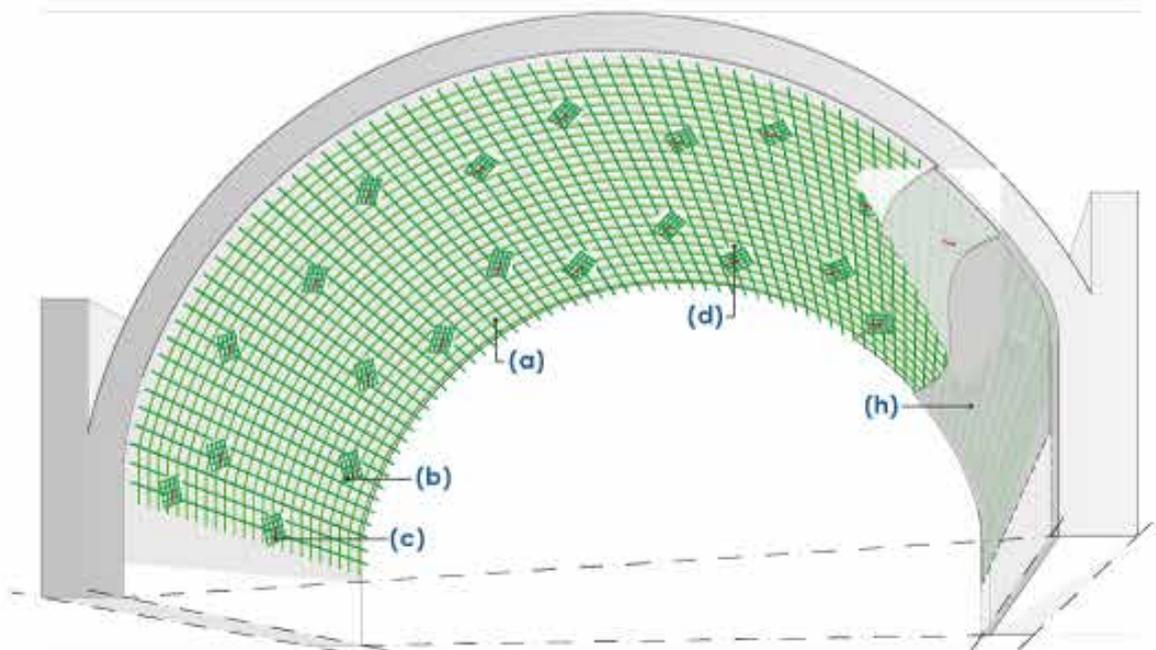
LIST OF DRAWINGS

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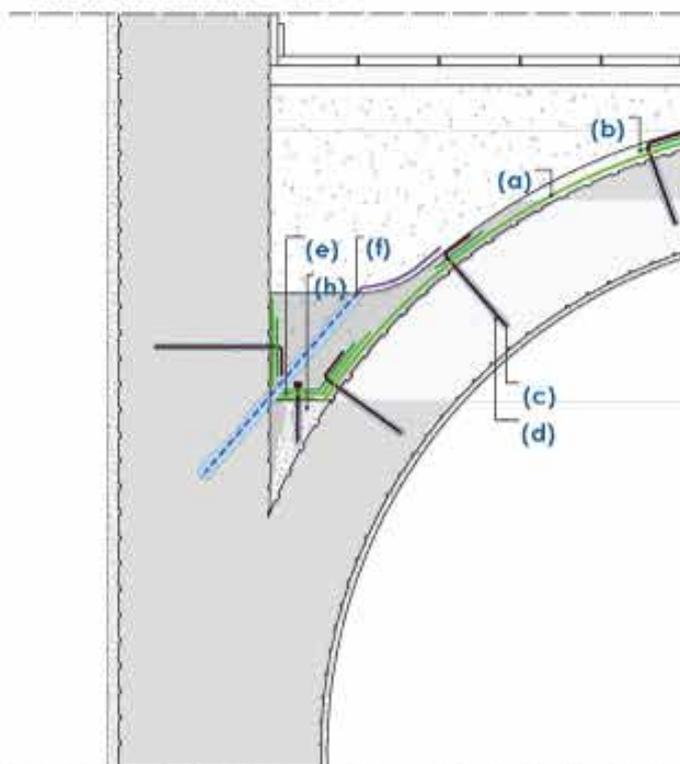**E-mail:** export@fibrenet.it **/ Phone:** +39 0432 600918

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RI-STRUTTURA

VISTA ASSONOMETRICA ESTRADOSSO

(h)

(c)

(d)

(a)

(b)

VISTA PROSPETTICA INTRADOSSO

(d)

(a)

(h)

(b)

(c)

INTONACO ARMATO - SISTEMA CRM IN G.F.R.P.

CONSOLIDAMENTO VOLTE A BOTTE

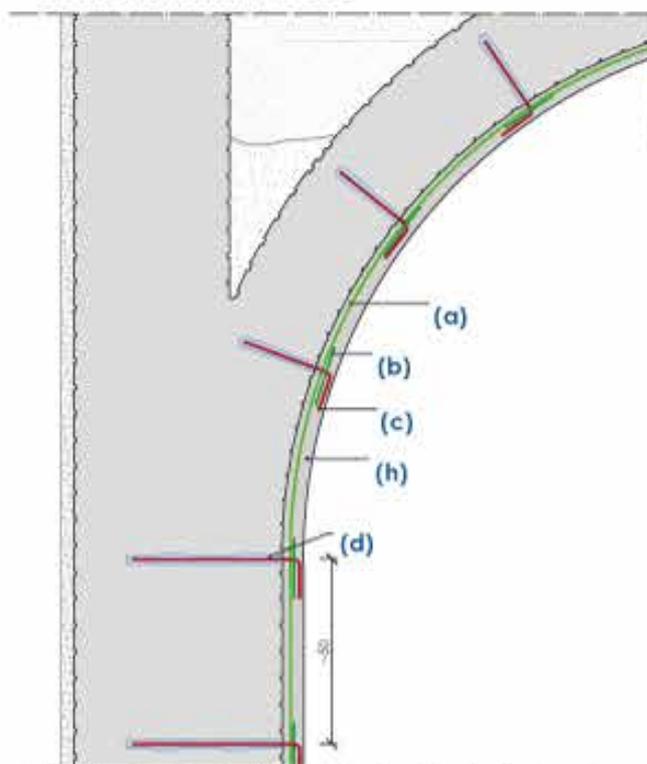INQUADRAMENTO GENERALE

$C\epsilon$

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SEZIONE ESTRADOSSO

SEZIONE INTRADOSSO

DESCRIZIONE DELLE LAVORAZIONI

Il sistema RI-STRUTTURA utilizza la tecnica dell'intonaco armato all'estradosso e/o all'intradosso della volta con l'applicazione di reti, connettori ed accessori in GFRP abbinati a malte a base calce con spessori ridotti (circa 3 cm).

La connessione dei due intonaci rinforzati è ottenuta attraverso la posa di elementi ad "L" in GFRP previsti in ragione di 4/6 al m² e disposti secondo uno schema a quincunce; la continuità dei connettori è garantita con una sovrapposizione non inferiore a 10 cm. Nel caso di intervento all'estradosso prevede la rimozione del pavimento, del sottofondo e del riempimento fino a raggiungere la quota del rinfianco. L'asportazione del riempimento deve avvenire in maniera uniforme e simmetrica rispetto alla chiave, altrimenti si consiglia di centinare adeguatamente la volta.

Successivamente è prevista la pulizia meccanica della superficie con aspirazione al polvere e parti incoerenti; valutare l'eventuale bagnatura della superficie e la posa in opera del successivo rinzaffo.

Le lavorazioni per la realizzazione del rinforzo si articolano nelle seguenti fasi:

- Posa sul piano dell'intonaco di rete **FBMESH** e di elementi angolari **FBANG** con sovrapposizione di 15/20cm ai giunti e negli spigoli.
- Tracciamento sui conci della posizione dei fori e loro realizzazione;
- Iniezione dei fori precedentemente realizzati con ancorante chimico bi-componente **INTEGRA FIXA VINYL15** e posa di fazzoletto **FB-FAZ 33X33T96AR** con connettore **FBCONL**. Procedere con l'applicazione dell'intonaco di malta **MATERIA/EPOCA** con spessore di 3cm.

NOTE:

In funzione delle condizioni termoigrometriche si consiglia la bagnatura del supporto il giorno prima della posa dell'intonaco.

E' possibile incrementare il numero di connessioni in funzione delle esigenze di calcolo.

IDENTIFICAZIONE MATERIALI CRM

- (a) **FBMESH** (Rete preformata in GFRP)
- (b) **FBFAZ33X33T96AR** (Fazzoletto di rete preformata in GFRP)
- (c) **FBCONL** (Connettore a "L" preformato in GFRP)
- (d) **INTEGRA FIXA VINYL 15** (Ancorante chimico bi-componente)
- (e) **FBANG** (Elemento in GFRP preformato ad angolo retto)
- (f) **FB-TUP10-VAR1A/2A** (Barra in materiale composito fibrorinforzato)
- (g) **FB-HBAR10** (Barra elicoidale in acciaio inox austenitico)
- (h) **MALTA MATERIA/EPOCA** (impasto premiscelato ecocompatibile a base calce)

10
CONCLUSIONS

The twenty-year experience of scientific research, combining experimental investigations and numerical simulations, has deepened the understanding of the CRM System as a reinforcement technique for existing masonry structures. In particular, the systematic adoption of the modular approach for experimental tests, and the multilevel approach for numerical analysis, has allowed for resource optimization and has proven to be very effective in addressing and understanding the various aspects that must be taken into consideration at different scales in the study of this consolidation system. Obviously, the research is ongoing and other aspects are currently being analyzed (such as the combination of actions or the effects of the dynamic nature of seismic action). However, the path taken and the research experience have clearly demonstrated that understanding the behavior of structures reinforced with CRM is a complex issue to tackle, which must extend from the scale of materials and individual collapse mechanisms to the behavior of structural elements and entire buildings, carefully evaluating the fundamental role of construction details. The existence of adequate studies aimed at understanding the various aspects is an indispensable condition for guiding the design choice towards reliable solutions to be adopted in building practice. To address these challenges, numerous doctoral scholarships and research grants have been and are still being awarded with the fundamental contribution of Fibre Net, as well as the significant number of Bachelor's and Master's theses focused on the experimental research of the company, past and present, in order to maintain the evolution of the System over time, not only from an experimental perspective but also from an applicative one.

11
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NOTES

113

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