**Deliverable D 4.1**

**Specification for laboratory specimens and testing strategies on walls**

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WORKPACKAGE 4: Optimization of design for vertical elements

Partners: UNIPD, ITAM, NTUA, POLIMI, UMINHO, UPC, CDCU, S&B, ZRS, MONU  
Leader: BAM

PROJECT N°: 244123  
ACRONYM: NIKER  
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COORDINATOR: Università di Padova (Italy)  
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Small or medium scale focused research project  
THEME: Environment (including Climate Change)
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1 INTRODUCTION AND GOALS OF THE WORK PACKAGE

The structural response of historical buildings to earthquakes is manifold and complex. This response does not only depend on the type of earthquake wave, amplitude, frequency and ground but also strongly depends on the type of structure and its design and performance features. In order to break down the complexity of the response inherent within different types of historical buildings, the concept of this project foresees to describe in a first step single structural building elements and in a later step the response of model buildings towards various load scenarios. Work packages 4 and 5 are therefore dedicated to vertical and horizontal structural elements. Work package 4 studies in depth the structural behaviour of masonry consisting of stone, brick and earth blocks (adobe) as well as monolithic walls such as cob and rammed earth.

The main objective of this work package is to define a general strategy for strengthening the previously described vertical elements by different techniques for in-plane or out-of-plane loading scenarios. This goal is embedded in strategies for improving the overall response of historical structures in the course of an earthquake event, which will follow in the subsequent work packages. The focus of WP4 is laid on minimum intervention techniques considering that many historical structures already sustained damages by previous earthquakes and/or are partially affected by other damaging processes. In detail the following goals are envisioned:

- Definition of adequate and feasible intervention methods for vertical structural elements related to the catalogues and requirements described in WP3.
- Definition and improvement of laboratory procedures for evaluating the intervention methods and specifications for laboratory specimens.
- To carry out the necessary tests to characterize the experimental behaviour of original and strengthened masonry walls and pillars, in order to obtain information on the system performance and the main constitutive laws relevant for modelling.
- To numerically simulate the experimental behaviour and perform parametric assessment to define critical mechanical parameters or define optimized design procedures.

The work plan (see Annex I – “Description of Work”) foresees two tasks in work package 4. Task WP4.1 is dedicated to laboratory experiments where masonry of stone, brick and adobe (earth blocks) as well as monolithic/homogeneous earthen walls will be studied under different load types (in form of in-plane and out of plane loading tests). In a second step low impact intervention techniques will be investigated on pre-damaged masonry specimens in the laboratory and in-situ in the field. These techniques will include grouting with compatible mortars, re-pointing/reinforced re-pointing, anchoring with glass fibre and steel elements and FRP-SRP/SRG applications. In task WP4.2, numerical modelling will be performed in order to simulate the experimental behaviour of the elements investigated in WP4.1. This is necessary in order to calibrate the models for the composite inelastic behaviour of the structural elements consisting of different construction materials and for the various intervention techniques applied in WP4.1. A numerical parametric assessment will be carried out on the calibrated models to define critical mechanical parameters, to highlight the limits of the proposed strengthening method and to support the optimization of the dimensioning and the design of the intervention technique at hand.

Deliverable 4.1 will outline the strategy for the laboratory and field experiments performed in task WP4.1. This concerns the different kinds of materials as well as type and specifications of specimens and experiments. However, materials and constructions for vertical elements addressed in WP4.1 represent only a small selection of the vast variety of different building techniques used in the past. Nevertheless, the selection of materials (stone, brick and earth) and the entailing construction types included in WP4 is/provides a representative range of structural categories with their respective behaviour under load, allowing at least general conclusions about the response within a structural category.
2 VERTICAL ELEMENTS UNDER EARTHQUAKE LOAD

2.1 Material properties

The response of vertical elements under earthquake loads depends on many factors. Crucial parameters derive from the type of construction and the materials being used. With respect to mechanical strength (e.g. compressive strength), the three groups of historical materials investigated in the project form three more or less distinctive categories with the following strength characteristics in terms of compressive strength:

Earth < brick < stone

A similar pattern is valid for their elastic behaviour. Overlaps between the categories are possible, since it is inherent within natural products (i.e. natural stone and earth) and early production technologies (i.e. brick) to yield a vast variety of characteristics.

2.1.1 Masonry materials and masonry

Vertical masonry elements are composite, non-homogeneous materials consisting of blocks (i.e. stone, brick or/and adobe) and mortar, which binds the masonry blocks together. Though masonry structures consisting of blocks alone were built, the majority of the historical masonry elements are jointed together by a mortar. In terms of structural behaviour, critical aspects in masonry can be the weakness of its mortar and the weakness of the bond between mortar and masonry units. Under dynamic loading, the cracking of masonry along the mortar joints allows energy dissipation to take place. The dissipation capacity of existing buildings is often affected by pre-existing cracks, moisture and damage. The earthquake evaluation processes provided by building codes such as Eurocode 8 often overestimate the safety of historical structures.

2.1.1.1 Mortar

Prior to industrialization, the main binding material in mortars consisted of lime, gypsum and earth. In the 19th Century, other binders such as Roman cement, Portland cement, slag cement or aluminate cement were developed. Gypsum as a binder for masonry mortar was predominantly used in ancient times, sometimes together with lime. Later, it was mostly utilized as a plaster or stucco material. Lime binders can be divided into high lime (including lime hydrate) and hydraulic lime. High lime mortars harden mostly by carbonation, which is a very slow process and, in case access to atmospheric CO₂ is blocked, not always completed. Hydraulic lime contains either calcium silicates/calcium aluminates (natural hydraulic lime) or pozzolanic components have been added. In both cases the hardening is controlled not only by carbonation but also by hydraulic reactions. A pozzolanic material which was added to lime putty mortars to produce a hydraulic binder was brick powder (in Italy also known as “cocciopesto”) and was frequently used by the Romans. Very clear pozzolanic reactions between lime and brick powder were detected from samples taken from ancient monuments in Italy and other European countries (Binda.L., Baronio, G., 1984a, Binda,L., Baronio,G., 1988a). After this study new mortars were reproduced in laboratory based on hydrated limes and their mechanical and physical behaviour tested for a duration of more than three years (Baronio, Binda &al., 2000) Earth mortars consist of a sand and silt framework with the clay fraction as binder. Earth mortars are considered as the weakest materials, though they can be equal or higher in strength than high lime mortars. In the past, lime has been added to earthen mortars in order to increase their strength. The exact strength of mortars depends on many variables. However, each binder type shows typical characteristics as shown in Table 2.1.

2.1.1.2 Masonry block materials

Stone

Masonry block materials show an even greater variety in properties, with natural stone being the material with the broadest value range. Natural building stone is generally categorized in three
main groups, according to origin: Magmatites, sediments and metamorphites. Table 2.2 gives an approximate range of values for compressive and flexural strengths and modulus of elasticity of the different groups of natural stone. It has to be noted that especially sedimentary and metamorphic stones can show anisotropy in their physical and mechanical properties. Many types of sediments, such as sandstone and limestone, are layered whereas metamorphic stones show schistosity causing different mechanical and physical properties parallel or perpendicular to the foliation.

Tab. 2.1 - Some characteristics of different types of mortars (Schäfer & Hilsdorf 1993; Houben & Guillaud, 1994; Müller et al., 2008; Magalhães & Veiga, 2009; Schubert, 2009).

<table>
<thead>
<tr>
<th>Type of binder</th>
<th>Compr. strength (MPa)</th>
<th>E-modulus (GPa)</th>
<th>other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth</td>
<td>0.5 - 3</td>
<td>&lt; 1</td>
<td>very susceptible to water, high hardening by drying</td>
</tr>
<tr>
<td>High lime and lime hydrate</td>
<td>0.5 - 3</td>
<td>1 - 5</td>
<td>susceptible to water, moderate to high shrinkage, slow setting by drying, very slow hardening by carbonation</td>
</tr>
<tr>
<td>Hydraulic lime</td>
<td>2 - 10</td>
<td>5 - 15</td>
<td>stable in water, moderate shrinkage, slow hardening by hydraulic reaction and carbonation</td>
</tr>
<tr>
<td>Roman cement</td>
<td>5 - 20</td>
<td>stable in water, low shrinkage, very fast setting and slow hardening by hydraulic reaction</td>
<td></td>
</tr>
<tr>
<td>Portland cement</td>
<td>10 - 50</td>
<td>20 - 30</td>
<td>stable in water, low shrinkage, fast setting and fast hardening by hydraulic reaction</td>
</tr>
<tr>
<td>Gypsum</td>
<td>2 - 15</td>
<td>&lt; 5</td>
<td>susceptible to water, fast setting and hardening (demi hydrate), slow setting and hardening (anhydrite)</td>
</tr>
</tbody>
</table>

Tab. 2.2 - Mechanical properties and porosity of natural stone (Grimm, 1990; Warnecke, 1998; Schubert, 2009).

<table>
<thead>
<tr>
<th>Type of binder</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
<th>E-modulus (GPa)</th>
<th>Porosity vol-%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magmatite</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granitic rocks</td>
<td>80 – 300</td>
<td>10 – 30</td>
<td>35 – 80</td>
<td>0.1 – 2</td>
</tr>
<tr>
<td>Basaltic rocks</td>
<td>160 – 400</td>
<td>15 – 25</td>
<td>50 – 100</td>
<td>0.7 – 29</td>
</tr>
<tr>
<td>Volcanic tuff</td>
<td>5 – 40</td>
<td>1 – 4</td>
<td>4 – 10</td>
<td>20 – 50</td>
</tr>
<tr>
<td>Sediment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandstone, siliceous</td>
<td>60 – 250</td>
<td>7 – 20</td>
<td>10 – 70</td>
<td>1 – 29</td>
</tr>
<tr>
<td>Sandstone, carbonate, clayey</td>
<td>15 – 150</td>
<td>3 – 15</td>
<td>5 – 30</td>
<td></td>
</tr>
<tr>
<td>Limestone, porous</td>
<td>20 – 90</td>
<td>5 – 8</td>
<td>5 – 20</td>
<td>0.1 – 35</td>
</tr>
<tr>
<td>Limestone, dense, dolomitic</td>
<td>80 – 90</td>
<td>6 – 15</td>
<td>15 – 80</td>
<td></td>
</tr>
<tr>
<td>Metamorphite</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gneiss</td>
<td>70 – 260</td>
<td>8 – 30</td>
<td>25 – 80</td>
<td>0.5 – 4</td>
</tr>
<tr>
<td>Marble</td>
<td>40 – 300</td>
<td>6 – 15</td>
<td>15 – 80</td>
<td>0.4 – 2</td>
</tr>
</tbody>
</table>

Natural stone shows as well a high variability in porosity (Tab. 2.2). Porosity is crucial concerning durability. Porous stone is usually more susceptible for degradation mechanisms. Masonry of historical buildings or archaeological objects may suffer from degradation caused by environmental influences and in severe cases leading to reduced cross sections of walls by subsequent back weathering (Fig. 2.1). But porosity is also linked to the mechanical properties of a stone: an increase in total porosity in most cases reduces mechanical strength (exceptions are travertine and some basaltic lava, which can have high porosity values due to a large number of macro pores and high strength). Another effect is that natural stone of different porosity can absorb corresponding amounts of water by capillary action. However, with increasing moisture content porous materials show generally an inverse strength behaviour, meaning that mechanical strength decreases with an increased moisture content. This has to be considered in masonry exposed to uprising dampness, often encountered in historical or archaeological structures. Generally speaking, the mechanical performance of porous stone is more affected by moisture than that of stone with a very low porosity.
Brick (clay brick)

The terms ‘brick’ or ‘clay brick’ are used here meaning fired clay and silt rich earthen blocks.

Brick, in particular if fired in field kilns, can show a similar variation in properties, though not as wide as those of natural stone. Mechanical properties and porosity are strongly influenced by the composition and processing of raw materials, by the shaping process and by the firing temperature, which in field kilns was gradually decreasing from the centre, where the fuel was burnt, to the outside. The temperature gradient within a field kiln yielded therefore also brick batches of different strength behaviour and porosity ranging from unfired/weakly burnt to completely sintered bricks (Egermann & Mayer, 1989).

Figure 2.2 gives an example of the compressive strength of historical bricks from the 19th Century fired in a field kiln. The bricks are from the same masonry of a historical building which was partially demolished. The graph demonstrates the huge variety in strength and porosity (apparent density) but not showing even lower strength data of below 10 MPa of manually formed and weakly fired bricks (Fernandes & Lourenco, 2007).

The modulus of elasticity of hand formed bricks fired in field kilns reach up to 10 GPa. Higher values can be reached by bricks, which were extruded (Egermann & Mayer, 1989). Bricks usually show a more or less anisotropic mechanical and physical behaviour caused by their internal texture due to the manufacturing process. In particular extruded bricks can show considerably different strengths parallel and perpendicular to the direction of extrusion.

The porosity of historical bricks lay roughly within the range of 20 to 30 vol.-%. As it is with natural stone, bricks tend to be reduced in strength properties when their moisture content increases. If a masonry structure is affected by uprising dampness the strength of the masonry is therefore be reduced towards the masonry base with higher moisture content.

Extensive research was carried out in Italy in the past decades and especially in Milan on the physical, chemical and mechanical characteristics of old and ancient bricks compared to new extruded bricks used for restoration and on their durability to salt crystallisation (Baronio & Binda, 1982) (Binda & Baronio, 1984b).
Stress-strain behaviour under compression of brick masonry prisms built with different types of mortars (hydrated lime, lime-cement and high-strength mortars) was studied together with durability to salt crystallisation since 1985 (Binda & Baronio, 1985), (Binda, Fontana & al., 1988b), (Binda, Fontana & al., 1991) and some of the results were adopted by the Canadian Recommendations for existing masonry structures.

The first studies on strengthening of brick masonries with epoxy resins and hydraulic grouts were carried out starting from 1988 by L. Binda and G. Baronio within a European Jumelage Contract together with the LGPC in Paris and the University of Athens (Binda, Baronio & al., 1988b), (Binda & Baronio, 1989), (Binda, Baronio & al., 1993a).

Adobe (earth blocks)

The terms ‘adobe’ and ‘earth blocks’ will only be used here for the description of building blocks made from earthen materials, which are air dried. Other synonymous terms, such as ‘mud brick’, ‘sun backed brick’ or ‘unfired brick’ often mentioned in literature will not be used.

Though adobe is a widely utilized building material since prehistoric times it represents also a type of masonry block which yields the lowest strength values. Typical values for compressive strength of historical adobe, which were not stabilized with lime, cement or other materials, are in a range from 1 MPa up to 5 MPa (Roehlen & Ziegert, 2010). The modulus of elasticity measured on modern earth blocks with similar compressive strength and particle size distribution as historical adobe is in the range of 0.4 to 2 GPa. Compared to some building stones or bricks, adobe shows only moderate to low anisotropic effects towards its mechanical and physical properties.

In general, earth reacts much stronger towards different moisture contents than any other porous mineral building material. Usually adobe shows much higher strength in dry state and can show very low strength when completely water saturated. The extreme would be a complete de-cohesion of the earth, which can take place when very high water contents are reached and the earth has a high content of sand and coarse silt sized fractions.

Figure 2.3 shows the compressive strength of three earth blocks under different moisture contents in equilibrium with the respective relative humidity, the samples were stored in. It clearly can be seen that variation in the moisture content can have a considerably big impact on material strength. One of the three tested adobes (in green colour, Fig. 2.3) loses half of its strength, when the moisture content is increased from 1.9 to 4.3 mass-% (which corresponds to a relative humidity change from 53 to 97 %). The porosities of adobe can be, depending on the fibre content, very high and reach values starting from 20 to up to 50 vol.-%.
Compressed earth blocks (CEB)

Compressed Earth Blocks (CEB) are made from soil, which has been filled into a mould and compacted with a hand-operated or motorized hydraulic press. The earthen material is soil moist when compacted and usually stabilized with cement, lime or gypsum. CEBs are more uniform in size than adobe and can be handled and stacked after compaction.

Though compressed earth blocks have been widely used since the 1950s, in the context of older historical buildings they are sometimes used for repair works, in particular in Northern Africa. CEBs show usually a lower mechanical strength than adobe. Their compressive strength depends on the compaction pressure and the amount of stabilizer. Usually CEBs need to have a finer grained particle size distribution. Typical values for compressive strengths range between 0.5 and 2.5 MPa and the modulus of elasticity between 0.1 GPa and 0.3 GPa.

2.1.2 Composite characteristics of masonry

An exhaustive description of masonry types is given in Deliverable 3.1 on page 24 to 52. The main parameters governing the structural behaviour of historical masonry walls, i.e. load bearing and deformation, include:

- Compressive and tensile strength of block and mortar and the constituent masonry
- Bond between block and mortar
- Type of masonry (regular, irregular, one leaf, multi leaf, …)
- Geometrical factors (i.e. slenderness)
- In multi leaf masonry: Bond between the single leaves and number and type of leaves

The list above shows, that the characteristic mechanical properties of masonry depend on many combinations of factors leading to a vast variety of data. On the other hand, standard calculation approaches for describing the structural behaviour of masonry are usually limited to modern regular masonry bond with very well characterized materials (e.g. modern bricks, calcium silicate, concrete blocks, aerated concrete blocks, etc.) and performed within their elastic range. The wide variety of historical masonry constructions and the materials usually used do not easily allow the application of the calculation models prescribed in the actual standards and regulations (e.g. Eurocode 6).

Though for regular historical one leaf masonry the actual calculation models for load and deformation behaviour may be applied, but usually critical data for historical materials are lacking and/or
many simplifications (e.g. homogeneity of materials, bond between blocks and mortar, eigenstress, transversal constraint, etc.) have to be made, leading to possible under- or overestimation of the load bearing capacity and deformation (Neuwald-Burg & Bohne, 1999). The load behaviour of historical irregular and/or multi leaf masonry is only partially covered by calculation approaches (Wenzel et al., 2000) taking into account many assumptions and simplifications leading to unreliable results.

An extensive research on the influence of mortar joints on the compressive strength and stiffness of masonry was carried out on Byzantine masonries in Italy and in other European countries and masonry prisms with thick mortar joints were reproduced following the composition of the studied mortars. The stress-strain behaviour under compression of the prisms at different ages was studied (Binda et al., 1999). The role of mortar joint thickness in the behaviour of brick masonry under compression is also described in (Binda et al., 2005).

For both, regular and irregular historical masonry experimental results concerning strength and deformation behaviour are needed, not only from the single materials but also from masonry elements under static and dynamic loading. The data to be acquired also with relevance to earthquake loading include mainly:

- Compressive strength of masonry and single materials as well as deformation behaviour
- Building block – mortar bond strength
- Tensile strength of mortar
- Shear strength of masonry

For numerical models further parameters are needed such as

- Adhesive shear strength
- Bond strength in tension
- Tensile strength of the stone

The experimental results will be used as input data for numerical modelling of the masonries in question.

2.1.3 Monolithic/homogeneous vertical elements

Two types of homogeneous (monolithic) earthen wall constructions will be considered in this work package: Rammed earth and cob. The structural behaviour depends mostly on the material characteristics of the earth used and the geometry of the wall element. Both terminologies describe not only specific types of materials but also unique construction techniques.

2.1.3.1 Rammed Earth

Rammed earth is a special construction technique utilizing formwork for the construction. It was/is practiced on all continents and known as pisé in French, tapial in Spanish, taipa in Portuguese, terra battuta in Italian and Stampflehm in German. Rammed earth shows two significant material characteristics:

- A fairly low moisture content (‘soil-moist’) when filled into the formwork and
- a wide, poorly sorted particle size distribution ranging from clay to gravel sized fractions.

Both, low moisture content (usually below the plastic limit of the earth) and the specific particle size distribution ideally allow a high compaction of the earth inside the formwork. Size and construction of the formwork depends on local tradition. Usually smaller units build in wood were used because of better handling and lower weight. Continuous formwork was less common. In the course of building a wall the formwork was subsequently lifted horizontally, vertically (one course) and horizontally again (and so on).
Due to the construction method a rammed earth wall is divided into sections reflecting the original dimension of the formwork unit. Depending on the shrinkage behaviour and original water content of the fresh earth, vertical and horizontal shrinkage joints may appear (Fig. 2.4). In this case, the entire rammed earth wall cannot strictly be described as 'monolithic'.

Local traditions included also adding lime to the earth as documented with many rammed earth buildings in Spain and Portugal (Jaquin et al., 2007; Sebastián & Cultrone, 2010). This was done by mixing lime and earth beforehand or by filling alternating layers of lime and earth into the formwork. Between the formwork lifts, often stones or bricks were placed. In some instances straw layers are added in case of earth with a high shrinkage. The straw helps to prevent that shrinkage joints spread into the rammed earth course above. As a consequence of the construction technique rammed earth has a distinctive horizontally layered texture.

The strength values depend on many factors (Vargas-Neumann, 1993): granulometry of the earth, moisture content, compaction, fibre content and amount of additions (lime …). These factors define also the apparent density and porosity, respectively. Typical values for the apparent density of historical rammed earth not stabilized with lime range between 1.6 and 2.3 kg/dm³. Corresponding compressive strength values lay within a range of 1.5 and 3.0 MPa (Roehlen & Ziegert, 2010). Historical rammed earth stabilized with lime can reach apparent densities of 1.7 to 2.1 kg/dm³ and compressive strength values of 2.0 to 11 MPa (Jaquin et al., 2007; Sebastián & Cultrone, 2010). Despite the apparent layered texture of rammed earth the mechanical properties seem not to be distinctively anisotropic. Laboratory studies showed that compressive strengths and E-modules measured parallel and perpendicular to the layering differed only within a range of 10 % (Bui & Morel, 2009). However, rammed earth with distinctive layers of fibres might show differences in the two directions.

The load behaviour of rammed earth was described by Dierks & Stein (2000) analogue to in-situ cast concrete. This was shown by short term compression tests of rammed earth and concrete specimen. However, against a complete analogue speaks the type of binding (clay minerals in case of earth, cement hydrate phases in the cement paste of concrete) and the shrinkage joints in rammed earth walls (Schröder, 2010).

Fig. 2.4 - Rammed earth construction in a vernacular house in Chile (left) and a rammed earth wall section of a historical building in Elche, Spain (right). The left image illustrates vertical and horizontal shrinkage joints outlining the formwork lifts (Source: U. Müller).
Concerning susceptibility of rammed earth towards the influence of water, the same can be stated as for adobe.

2.1.3.2 Cob

Similar to rammed earth, the term cob describes not only a material but also a construction technique. The material 'cob' is a mixture of earth and straw or other plant fibres. The largest size of the earth usually does not exceed the sand fraction. The fibre content is quite high (up to 25 kg/m³ earth); the fibre length usually is 30 to 50 cm. The earth plasticized with water is mixed with straw in an elaborate process. Walls of cob are built without formwork. The prepared earth/fibre mix is stacked freehanded to a linear mound to a height of ca 80 cm and then cut with a spade into a wall segment (Fig. 2.5).

Due to the high fibre content the apparent density is rather low (and porosity high). Typical density values of historical cob are within a range of 1.2 to 1.7 kg/dm³ (Ziegert, 2000; Schröder, 2010). Corresponding compressive strength is between 0.5 to 1.5 MPa. The modulus of elasticity is the lowest of all the earthen materials used for structural elements. Typical values are within 0.2 to 0.5 GPa. The original structural behaviour of cob buildings can be impacted by many environmental influences. Increased water content (due to uprising damp or faulty roof) not only lowers material strength but can also initiate rot of the fibres. The high fibre content enables insects or rodents to burrow deeply in cob walls. All these factors impair the overall structural behaviour of cob walls.

2.2 Load scenarios for vertical elements with respect to experiments

2.2.1 Earthquakes and seismic waves

Earthquakes movements are the result of the movement of tectonic plates. Tuzo-Wilson (1965) introduced the concept of plates and considers three fundamentally different types of displacement: spreading at ocean ridges, subduction at ocean trenches, and transformational boundaries. Thus, seismic activity is produced at seismic regions and is concentrated at tectonic boundaries. The intensity of shaking is measured as the predominant magnitude. A relative measuring system named the Modified Mercalli intensity scale (MMI) depends upon observations of human reaction and structural damage (Tolles et al., 1996) and ranges on a scale from I to XII. Other systems measure the amplitudes on seismograms on logarithmic scale. The most common of these scales is the Richter scale (M) defined as: $M = \log A - \log A_0$ (A: Max amplitude, $A_0$: Standard amplitude). Seismologists measure magnitudes in terms of different types of ground motions (seismic waves), which are substantially important for engineering purposes.

Seismic movements can be categorized into four types of waves, which travel at speeds ranging from 3 to 15 km/s: P and S waves, also grouped as body waves, and Love and Rayleigh waves, grouped as surface waves (Fig. 2.6A). Body waves move through the interior of the earth, as op-
posed to surface waves, which travel near the earth's surface. A P wave (compression wave) shakes the ground back and forth in parallel to the direction of travel of the wave energy. An S wave (shear wave) shakes the ground back and forth perpendicular to the direction of travel of the wave energy. A Love wave has a horizontal motion that is transverse to the direction of wave propagation. A Rayleigh wave causes the ground to shake in an elliptical motion, with no transverse or perpendicular direction. Waves produced during an earthquake propagate through the earth with different velocities. Therefore accelerograms from some distance away from the epicentre consist of two separate trains of oscillation, one for each type of the waves (example of P- and S-waves Figure 2.6B). Strong ground motions must be attributed to multiple reflection and refraction at irregular and sometimes diffuse geologic interfaces.

![Seismic Waves](image)

**Fig. 2.6 - A) Types of seismic waves. Two different types of waves are produced during an earthquake: 1) Body, P(rimary) and S(secondary), waves characterized by the movement through the earth, and 2) Surface, Rayleigh and Love, waves characterized by the movement near the surface. B) Accelerogramme showing schematic types of earthquake wave motions (P wave, S wave). Both images from Nejati (2005).**

### 2.2.2 Seismic loading and structural response of vertical elements

The principal cause of earthquake-induced damage on structures is horizontal ground shaking. As the earth vibrates, all buildings on the ground will respond to the vibration in varying degrees. Earthquake induced accelerations, velocities and displacements can damage or destroy a building unless it has been designed and constructed or strengthened to be earthquake resistant. Therefore, the effect of ground shaking on buildings is a principal area of consideration in the design of earthquake resistant buildings. Seismic design loads are extremely difficult to determine due to the random nature of earthquake motions and interdependencies with the local geological conditions. However, experiences from past strong earthquakes have shown that reasonable and prudent practices can keep a building safe during an earthquake.

A comprehensive overview of failure mechanisms of walls and pillars under earthquake load is given in Deliverable 3.1, page 52 to 64. The following section shall only recapitulate the most basic response mechanisms of vertical elements towards lateral load.

Seismic loading of a structure is caused by its inertia to a change in its state of motion in the case of horizontal movements of the earth. The force depends on the mass and geometry of the struc-
ture as well as the ground movement. Figure 2.7 illustrates different possible loading scenarios for structures under the influence of P, S and Love waves. The resulting lateral force or seismic load is represented by the force F as shown in Figure 2.7e. Depending on the direction of walls with respect to the direction of the horizontal ground movements, walls are subjected either to out-of-plane bending or to in-plane shear stresses. These forces cause the damage or collapse of a building. Additional vertical load effects are caused by S or Rayleigh waves inducing stress on beams, columns and floors. At certain instants of time the effective loads are increased, at others are decreased. It is to notice that factors such as maximum accelerations, type of building ground, usage, weight, stiffness and damping of the structure strongly influence the seismic loads (IAEE, 2004).

Fig. 2.7 - Seismic response of a building (from IAEE, 2004).

Optimization of design for walls and pillars D4.1

The horizontal seismic forces are reversible in direction. Vertical elements (i.e. walls, and pillars) bearing predominantly vertical loads under normal service conditions, have to carry extreme shear loads and horizontal bending during an earthquake (Fig. 2.8). If the building material is weak in tension such as earth, brick or stone masonry, cracking occurs which reduces the effective area for resisting the flexural moment, as shown in Figure 2.8. It shows that the strength in tension and shear is important for earthquake resistance.

Walls and masonry fail either due to in-plane or out-of-plane action where horizontal loads are usually transferred by in-plane action of walls. Figure 2.9 (left) illustrates failure mechanisms of wall elements due to in-plane movement. Three main failure modes are possible (Nejati, 2005): sliding (a in Fig. 2.9), flexural (b in Fig. 2.9) and shear (b in Fig. 2.9). Sliding failure describes the horizontal movement of a wall element on a brick course, mortar joint or other horizontal feature such as vapour barrier. Flexural failure occurs with cracking in the tension zone and compression at the opposite wall base, especially if vertical stresses due to dead loads are low. Shear failure is defined as a combination of tensile and compression stresses leading to typical diagonal cracks. Es-

Fig. 2.8 - Stress conditions in a wall element during service and earthquakes (from IAEE, 2004).

Fig. 2.9 - Failure patterns of wall/masonry elements due to in-plane (left) and out-of-plane actions (right). Both drawings from Nejati (2005).
Essentially two types of shear cracks can be observed in praxis: Joint cracks, where diagonal cracks run through mortar joints (in case of masonry) and diagonal cracks with cracks running through joints and blocks of masonry or through the material in homogeneous walls. Shear failure frequently occurs, if the height to length ratio of a wall is low. Out-of-plane forces cause walls perpendicular to the wave propagation to bend due to their inertia mass. Figure 9 shows crack patterns of walls due to out-of-plane loading.

Fig. 2.10 - Typical performance curve for a structure (from Ghobarah 2001).

Performance levels (i.e. collapse prevention), can be related to the following structural characteristics: Stiffness, strength and deformation capacity (Fig. 2.10). The displacements or drift limits are also functions of the structural system and its ability to deform (ductility). Design criteria may be established on the basis of observation and experimental data of deformation capacity. For example, near the collapse point, the drift limits of structural walls are different from a moment-resisting frame, which suggest that different structural systems will undergo unequal displacements.

### 2.2.3 Simulated earthquake motions

Earthquake simulations are usually performed by statistically applying horizontal inertia forces based on scaled peak ground accelerations. In most seismic design, earthquake loads are represented by the response spectrum of absolute acceleration. Earthquake ground motion synthesis plays a crucial role for the seismic design of masonry buildings (Shinouzuka et al. 1999), and the response spectra of a structure (displacement, acceleration and frequency) determines its capacity for sustaining strong earthquake ground motions (Fig. 2.11).

Currently in masonry building studies there is a trend towards displacement-based design (Torrealva, 2010; Tolles et al., 2000; Benedetti et al., 1998), which requires the earthquake loads to be represented by the response spectrum of relative displacement and the inelastic deformations can be accounted for by an equivalent damping (Fig. 2.11). There are many design situations, however, for which the earthquake actions need to be represented in the form of acceleration time-histories. The representation of the seismic hazard in the form of acceleration time-histories means that the hazard is defined in terms of all of the characteristics of the ground shaking. In addition to the amplitude and frequency characteristics that would normally define the elastic response spectrum, the energy and the duration of shaking are also considered.

These parameters can be considered to collectively represent the inelastic demand that the ground motion imposes on structures and therefore should actually be considered in all seismic hazard assessments. It is important that the acceleration time-histories employed in seismic design are consistent with the seismicity of the region and representative of the expected or design earthquakes.
Fig. 2.11 - A. Maximum accelerations of horizontal and vertical components of the BAM earthquake (Modified from Kiyono & Kalantari 2004). B. Base acceleration and frequency recorded at Calitri-Irpina earthquake 1980. Base acceleration in x-direction normalized to the peak value; signals acting in the orthogonal horizontal direction and along the vertical one are similar to it. Response spectra of the S and L base x-accelerations, normalized to their peak values. In both instances, major spectral amplifications occur for frequencies between 1 and 5 Hz. (Modified from Benedetti et al. 1998).
3 EXPERIMENTAL RESULTS ON VERTICAL ELEMENTS AND TEST PROCEDURES – STATE OF THE ART

3.1 Laboratory tests on stone masonry walls

3.1.1 Simple compression tests

Tomaževic (1992) carried out an experimental campaign on two layered rubble stone masonry walls, employing materials typical of the Slovenian region. Panels had an irregular texture and thick mortar joints. A cement-based grout was employed, without and with addition of hydrophobic additives to reduce the environmental effects and the capillarity activity. The injection operations were executed at an about constant pressure of 0.2 bar on bore holes spaced of 0.5-1.0 m. These holes were drilled on the mortar joints and wetted before the injection. Main results are reported in Table 3.1. Furthermore, the compressive strength of the injected grout is reported \( f_{gr} \). The tests exhibited the clear possibility to bind together the cracked parts of the stone masonry, obtaining a solid structure.

<table>
<thead>
<tr>
<th>Type of masonry</th>
<th>Description</th>
<th>( f_{pr} ) [N/mm²]</th>
<th>( f_{ww} ) [N/mm²]</th>
<th>( E ) [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncoursed stone, two layers, muddied sand</td>
<td>original</td>
<td>-</td>
<td>0.5</td>
<td>197</td>
</tr>
<tr>
<td></td>
<td>grated</td>
<td>33</td>
<td>1</td>
<td>825</td>
</tr>
<tr>
<td>Uncoursed stone, two layers, clean sand</td>
<td>original</td>
<td>-</td>
<td>0.7</td>
<td>390</td>
</tr>
<tr>
<td></td>
<td>grated</td>
<td>33</td>
<td>2.14</td>
<td>2744</td>
</tr>
</tbody>
</table>

Studies on the behaviour under compression of multiple leaf stone masonry walls were carried out by Binda on typical Italian masonries (Binda et al., 1991), (Binda et al., 1994), which lead to a classification of multiple leaf masonry (Binda et al., 2003a), which was also transferred to a database.

A wide series of in-situ tests was performed by Modena (Bettio et al. 1993; Modena and Bettio 1994) with the aim to compare the influence and the effectiveness of interventions through injections and jacketing. The considered panels mainly consisted of two-leaf masonry without any transversal connection. Different grouts were employed, mainly based on hydraulic lime and quick-lime, hydraulic lime with crushed bricks and cementitious additives (without any sulphate). The injection operations were realized at low pressure (0.5 bar) with a preliminary washing with water. The mesh was regular, considering 10 holes per square meter, and it was realized only in one side of the wall. The strengthening increased the compressive strength of the original masonry about 50 %. The results are summarized in Table 3.2. Elastic moduli are computed among different stress ranges \( E_1 \) between 0.20[N/mm^2] and 0.40[N/mm^2]; \( E_2 \) between 0.40[N/mm^2] and 0.80[N/mm^2]; \( E_3 \) between 0.80[N/mm^2] and 1.20[N/mm^2]).

During these studies also a new methodology to choose the best grouts for injection of multiple leaf masonries was proposed by Baronio and Binda. It was based on onsite investigation on the masonry section, recovering loose and poor material from the masonry section and filling them into transparent cylinders. The cylinders were brought to the laboratory where they were injected with different types of grout previously characterised and compared to the material coming from the interior of the masonry (Binda et al., 1992), (Binda et al., 1993b), (Binda et al., 1994). After the appropriate aging the cylinders were cut into two parts to study the penetration and distribution of the grout or were tested under compression in order to check the effects of the injection and the influence of the grout stiffness (Laefer et al., 1996).
Tab. 3.2 - Experimental results obtained by Modena and Bettio (1994), [N/mm²].

<table>
<thead>
<tr>
<th>Wall</th>
<th>Before strengthening</th>
<th>After jacketing</th>
<th>After injection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E₁</td>
<td>E₂</td>
<td>E₃</td>
</tr>
<tr>
<td>C₁ A</td>
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<td>130</td>
<td>70</td>
</tr>
<tr>
<td>B</td>
<td>180</td>
<td>150</td>
<td>10</td>
</tr>
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<td>C₂</td>
<td>470</td>
<td>240</td>
<td>130</td>
</tr>
<tr>
<td>A</td>
<td>580</td>
<td>500</td>
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<tr>
<td>B</td>
<td>800</td>
<td>430</td>
<td>250</td>
</tr>
<tr>
<td>C₃</td>
<td>450</td>
<td>320</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>870</td>
<td>370</td>
<td>90</td>
</tr>
<tr>
<td>B</td>
<td>490</td>
<td>900</td>
<td>130</td>
</tr>
<tr>
<td>C₄</td>
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</tr>
<tr>
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<td>370</td>
<td>90</td>
</tr>
<tr>
<td>B</td>
<td>2300</td>
<td>240</td>
<td></td>
</tr>
<tr>
<td>C₅</td>
<td>0.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C₆</td>
<td>550</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>10400</td>
<td>7900</td>
<td>6700</td>
</tr>
<tr>
<td>B</td>
<td>12300</td>
<td>10900</td>
<td>9400</td>
</tr>
<tr>
<td>C₇</td>
<td>9000</td>
<td>10400</td>
<td>8600</td>
</tr>
<tr>
<td>B</td>
<td>8000</td>
<td>9400</td>
<td>8400</td>
</tr>
<tr>
<td>R₁</td>
<td>A</td>
<td>800</td>
<td>1100</td>
</tr>
<tr>
<td>B</td>
<td>3500</td>
<td>2500</td>
<td>1.00</td>
</tr>
<tr>
<td>R₂</td>
<td>130</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>3400</td>
<td>2000</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>2400</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>19700</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Vintzileou and Tassios (1995) performed compression tests on three-leaf stone masonry with squared elements. The 8 specimens tested in compression had an overall dimension of 70x50x120cm; 2 of these were provided with transversal interlocking (samples 1, 3; Fig. 3.1), while any connection was inserted in the remaining specimens (samples 2, 4÷8; Fig. 3.3). The high void presence allowed the injection at a low pressure (0.07bar) of two cement grouts characterized by different compression strength, namely 30N/mm² (type A) and 13N/mm² (type B). The main damage involved sub-vertical cracks on the main side of the specimens but also vertical cracks at the interface between internal core and external leaves, at about the 50% of compressive strength. This caused wide out-of-plane displacements, as a consequence of a buckling effect, which led to a premature failure. The grout injection demonstrated the ability to delay this mechanism, which manifested close to the failure. Nevertheless, the injection of cement-based grouts caused also a significant increasing of the elastic modulus of strengthened panels. Finally, despite the widely different compressive strength of grouts, injected samples manifested similar compressive strength. The results are summarized in Table 3.3.

Fig. 3.1 - Specimens tested by Vintzileou and Tassios (1995).

Toumbakari and van Gemert carried out an experimental work, (Toumbakari and van Gemert 1997; Pisano 1999), on four multi-leaf stone masonry panels. Specimens, tested in laboratory, had overall dimensions of 60x120x40m. The external layers had calcareous stones and incoherent fragments in the central layer. Three different grout typologies were applied and injected at low pressure (0.08÷0.1bar) with a regular and dense mesh (holes at 20 ÷ 25 cm). Also the influence of transversal connections was investigated, since they were provided in one panel (specimen 3 (4 ÷ 5 per m²). The survey detected a percentage of about 15% of voids, allowing injection.
First damages occurred on main sides of samples and, only subsequently, cracks developed also on lateral side, at the interface between external layers and internal core. Furthermore, cracks mainly developed in mortar joints rather than in stones. The presence of transversal connections widely limits the out-of-plane deformation, even if the compression strength appeared very close to that of other samples. Finally, also the employment of grout with different strengths has a very limited influence on the overall behaviour of injected walls. The results are summarized in Table 3.4.

Vignoli performed several in-situ compression tests in some sites located in Tuscany (Pieve Fosciana, S. Anastasio, Pognana, Canova, Castelletto e Merizzo). These studies were also accompanied by on-site testing of the same masonries by the use of flat-jack tests carried out by the Politecnico di Milano team (L. Binda, G. Baronio and al.). Some specimens were loaded up to the elastic limit, then unloaded, to realize a strengthening intervention, and finally loaded up to the failure. Other samples, as reference specimens, were only loaded up to failure (samples in Pognana and S. Anastasio). The employed techniques were jacketing and grout injection, (Modena 1999). The masonry was mainly constituted by two accosted leaves with irregular stones and cobblestones. The overall thickness was ranging between 44÷70cm, while height and width ranged between 167÷205cm and 84÷101cm respectively. The applied techniques showed a considerable increase of both the strength and elastic modulus, as reported in Table 3.5 (R=Repaired; C=Compression).

Tab. 3.3 - Experimental results obtained by Vintzileou and Tassios (1995).

<table>
<thead>
<tr>
<th>wall</th>
<th>grout</th>
<th>$f_{wc0}$</th>
<th>$f_{wc,s}$</th>
<th>$f_{wc,s} / f_{wc0}$</th>
<th>$E_{wc0}$</th>
<th>$E_{wc,s}$</th>
<th>$\varepsilon_{wc0}$</th>
<th>$\varepsilon_{wc,s}$</th>
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<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>2.10</td>
<td>3.10</td>
<td>1.48</td>
<td>7000</td>
<td>6250</td>
<td>104</td>
<td>60</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>1.30</td>
<td></td>
<td>-</td>
<td>2706</td>
<td></td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>2.40</td>
<td>4.30</td>
<td>1.79</td>
<td>5000</td>
<td>5971</td>
<td>141</td>
<td>110</td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>1.60</td>
<td></td>
<td>-</td>
<td>4442</td>
<td></td>
<td>-</td>
<td>80</td>
</tr>
<tr>
<td>5</td>
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<td>1.70</td>
<td>4.20</td>
<td>2.47</td>
<td>5670</td>
<td>7778</td>
<td>286</td>
<td>120</td>
</tr>
<tr>
<td>6</td>
<td>B</td>
<td>1.35</td>
<td>4.05</td>
<td>3.00</td>
<td>5625</td>
<td>8438</td>
<td>58</td>
<td>100</td>
</tr>
<tr>
<td>7</td>
<td>A</td>
<td>-</td>
<td>3.70</td>
<td></td>
<td></td>
<td>15413</td>
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<td>8</td>
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<td>-</td>
<td>3.00</td>
<td></td>
<td></td>
<td>3333</td>
<td>-</td>
<td>90</td>
</tr>
</tbody>
</table>

Tab. 3.4 - Experimental results obtained by Toumbakari and van Gemert (1997).

<table>
<thead>
<tr>
<th>wall</th>
<th>$f_{gr}$</th>
<th>$f_{wc0}$</th>
<th>$f_{wc,s}$</th>
<th>$f_{wc,s} / f_{wc0}$</th>
<th>$E_{wc0}$</th>
<th>$E_{wc,s}$</th>
<th>$\varepsilon_{wc0}$</th>
<th>$\varepsilon_{wc,s}$</th>
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<td>1</td>
<td>6.4</td>
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<td>-</td>
<td>1.42</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>14.6</td>
<td>2.1</td>
<td>3.3</td>
<td>1.6</td>
<td>4400</td>
<td>4500</td>
<td>1.6</td>
<td>2.3</td>
</tr>
<tr>
<td>3</td>
<td>5.2</td>
<td>2.6</td>
<td>3.5</td>
<td>1.7</td>
<td>5800</td>
<td>4000</td>
<td>1.7</td>
<td>2.4</td>
</tr>
<tr>
<td>4</td>
<td>5.2</td>
<td>2.7</td>
<td>3.3</td>
<td>2.1</td>
<td>5200</td>
<td>1900</td>
<td>2.1</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Fig. 3.2 - Stress-strain relationship of tested specimens, (Modena 1999).
Valluzzi (2000) performed an experimental campaign on 16 three-leaf stone masonry panels to study the influence of different strengthening techniques, namely lime grout injection, repointing and insertion of transversal steel ties, firstly proposed in collaboration with the Politecnico of Milan (Binda, Modena et al., 1999b). In all cases, for grout as well as for mortar, lime-based materials were employed to ensure a better overall compatibility with historical materials. Nevertheless, two different types of admixtures were employed for the strengthening interventions. The experimental observation and results, reported in Table 3.6, demonstrate the capability of injections to improve the mechanical characteristics of the multi-leaf masonry. Particularly, the out-of-plane deformations could be limited and the compressive strength improved. Moreover, the elastic properties were only slightly influenced by the injection, without a significant increasing of the elastic modulus. Further techniques manifested a lower influence on the strength. Nevertheless, tying allowed to avoid brittle failure modes, while repointing increased the durability of the considered element. The combination of these induced the best results in terms of overall behaviour of the specimens. Finally, results confirmed that the employment of different grouts, having higher mechanical properties, does not influence significantly the overall strength of the walls.

Tab. 3.5 - Experimental results obtained by Vignoli, (Modena 1999).

<table>
<thead>
<tr>
<th>Intervention</th>
<th>σ_{ul}</th>
<th>σ_{max}</th>
<th>E_{k/3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pieve Fosciana</td>
<td>0.654</td>
<td>-</td>
<td>1468</td>
</tr>
<tr>
<td>(A) Jacketing</td>
<td>0.698</td>
<td>-</td>
<td>3051</td>
</tr>
<tr>
<td>Pieve Fosciana</td>
<td>0.505</td>
<td>-</td>
<td>1333</td>
</tr>
<tr>
<td>(B) Jacketing</td>
<td>0.788</td>
<td>-</td>
<td>5169</td>
</tr>
<tr>
<td>Poggiana</td>
<td>0.145</td>
<td>-</td>
<td>250</td>
</tr>
<tr>
<td>(E) Injection</td>
<td>0.205</td>
<td>-</td>
<td>4667</td>
</tr>
<tr>
<td>Poggiana</td>
<td>0.158</td>
<td>1.041</td>
<td>376</td>
</tr>
<tr>
<td>(D) Injection</td>
<td>0.348</td>
<td>1.596</td>
<td>3012</td>
</tr>
<tr>
<td>S. Anastasio</td>
<td>0.158</td>
<td>0.236</td>
<td>1921</td>
</tr>
</tbody>
</table>

Tab. 3.6 - Experimental results obtained by Valluzzi (2000).

<table>
<thead>
<tr>
<th>Panel</th>
<th>Intervention</th>
<th>f_{w,0}</th>
<th>f_{w,\sigma}</th>
<th>E_{w,0}</th>
<th>E_{w,\sigma}</th>
</tr>
</thead>
<tbody>
<tr>
<td>511</td>
<td>Injection</td>
<td>1.45</td>
<td>2.49</td>
<td>2390</td>
<td>2273</td>
</tr>
<tr>
<td>611</td>
<td>Injection</td>
<td>1.95</td>
<td>2.49</td>
<td>2029</td>
<td>3093</td>
</tr>
<tr>
<td>13H1</td>
<td>Injection</td>
<td>-</td>
<td>2.54</td>
<td>-</td>
<td>3992</td>
</tr>
<tr>
<td>112</td>
<td>Injection</td>
<td>1.97</td>
<td>2.57</td>
<td>1450</td>
<td>3449</td>
</tr>
<tr>
<td>812</td>
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<td>1559</td>
<td>2367</td>
</tr>
<tr>
<td>161</td>
<td>Injection</td>
<td>-</td>
<td>2.48</td>
<td>-</td>
<td>1223</td>
</tr>
<tr>
<td>2T</td>
<td>Tying</td>
<td>1.95</td>
<td>1.77</td>
<td>1504</td>
<td>1789</td>
</tr>
<tr>
<td>9T</td>
<td>Tying</td>
<td>1.65</td>
<td>1.34</td>
<td>2058</td>
<td>1932</td>
</tr>
<tr>
<td>11T</td>
<td>Tying</td>
<td>-</td>
<td>1.26</td>
<td>-</td>
<td>474</td>
</tr>
<tr>
<td>10RT</td>
<td>Repointing+Tying</td>
<td>-</td>
<td>0.88</td>
<td>-</td>
<td>110</td>
</tr>
<tr>
<td>12HT</td>
<td>Repointing+Tying</td>
<td>-</td>
<td>2.59</td>
<td>-</td>
<td>1336</td>
</tr>
<tr>
<td>141HR</td>
<td>Injecting+Repointing</td>
<td>-</td>
<td>2.14</td>
<td>-</td>
<td>1617</td>
</tr>
<tr>
<td>1711RT</td>
<td>Injecting+Tying+Repointing</td>
<td>-</td>
<td>3.06</td>
<td>-</td>
<td>1772</td>
</tr>
<tr>
<td>3R</td>
<td>Repointing</td>
<td>0.99</td>
<td>1.34</td>
<td>-</td>
<td>538</td>
</tr>
<tr>
<td>7R</td>
<td>Repointing</td>
<td>1.50</td>
<td>1.17</td>
<td>1863</td>
<td>1781</td>
</tr>
<tr>
<td>15R</td>
<td>Repointing</td>
<td>-</td>
<td>1.32</td>
<td>-</td>
<td>395</td>
</tr>
</tbody>
</table>
Toumbakari (2002) carried out an experimental campaign on nine three-leaf masonry specimens tested under compression. Four of these were realized using bricks, while stones were used for the remaining samples. The overall dimensions of all the specimens were 60x40x120 cm$^3$. Furthermore, different grouts were studied and injected, with similar injectability properties but different mechanical characteristics and reduced Portland cement content. The specimens were tested firstly without strengthening and, after the interventions; they were subjected again to compression up to failure. The experimental observations confirmed that the main failure mechanism, in both cases, is the consequence of the bucking of external layers. Furthermore, results, showed in Fig. 3.4 and presented in Table 3.7, highlight the capability of grout injection to improve the overall strength of the wall, even if the increase seems to be independent from the mechanical properties of injected grout. Differently, the stronger the grouts, the higher the increase in elastic modulus of masonry samples. Finally, the great influence of different employed materials is mainly due to the shear bond strength of the grout-substratum interface.

At the University of Minho, Oliveira and Lourenço (2006) carried out first tests on three-leaf stone masonry walls. These experiments are part of a wide ongoing experimental campaign to deepen the mechanical behaviour of this masonry typology. Furthermore, several strengthening techniques will be investigated. First results, presented in Tab. 3.8, are referred to unreinforced specimens and samples strengthened through the insertion of transversal GFRP ties. These results highlighted that the application of these ties allow sustaining a load increase of about 70% with reference to the unstrengthened conditions. Furthermore, the most typical failure mechanism, namely out-of-plane buckling of outer layer could be widely limited, allowing the development of vertical cracks, as in the case of a monolithic material.
Tab. 3.7 - Experimental results obtained by Toumbakari (2002).

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{\text{sec}0}$ [N/mm$^2$]</th>
<th>$f_{\text{sec},d}$ [N/mm$^2$]</th>
<th>$E_{\text{sec}0}$ [N/mm$^2$]</th>
<th>$E_{\text{sec},d}$ [N/mm$^2$]</th>
<th>$\varepsilon_{\text{sec}0}$</th>
<th>$\varepsilon_{\text{sec},d}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC1</td>
<td>2.41</td>
<td>3.00</td>
<td>156.4</td>
<td>140.4</td>
<td>2.42</td>
<td>3.55</td>
</tr>
<tr>
<td>BC2</td>
<td>2.09</td>
<td>3.33</td>
<td>101.8</td>
<td>112.2</td>
<td>2.23</td>
<td>3.55</td>
</tr>
<tr>
<td>BC3</td>
<td>2.28</td>
<td>3.66</td>
<td>114.4</td>
<td>117.0</td>
<td>2.31</td>
<td>3.14</td>
</tr>
<tr>
<td>SC1</td>
<td>2.02</td>
<td>3.25</td>
<td>720.4</td>
<td>1622.2</td>
<td>1.42</td>
<td>3.55</td>
</tr>
<tr>
<td>SC2</td>
<td>2.09</td>
<td>3.36</td>
<td>1138.7</td>
<td>1558.6</td>
<td>1.65</td>
<td>2.34</td>
</tr>
<tr>
<td>SC3</td>
<td>2.65</td>
<td>3.51</td>
<td>1374.8</td>
<td>1187.6</td>
<td>1.73</td>
<td>2.45</td>
</tr>
<tr>
<td>SC4</td>
<td>2.71</td>
<td>3.29</td>
<td>1443.3</td>
<td>1014.5</td>
<td>2.11</td>
<td>3.49</td>
</tr>
</tbody>
</table>

Tab. 3.8 - Experimental results obtained by Oliveira and Lourenço (2006).

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{\text{sec}}$ [N/mm$^2$]</th>
<th>Panel</th>
<th>$f_{\text{sec}}$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1W1</td>
<td>2.40</td>
<td>2W2</td>
<td>3.30</td>
</tr>
<tr>
<td>1W2</td>
<td>1.70</td>
<td>2W3</td>
<td>2.60</td>
</tr>
<tr>
<td>2W1</td>
<td>1.40</td>
<td>2W4</td>
<td>3.50</td>
</tr>
</tbody>
</table>

Binda et al. developed an experimental campaign parallel to a numerical investigation (Binda, Anzani at al. 2003b, Pina-Henriques and Lourenço 2003; Binda et al. 2006), in order to deepen the knowledge of both the stress distribution and load-transfer mechanisms on three-leaf stone masonry. The whole program involved compression tests also on each singular leaf, while the results of the compression tests on whole masonry panels are reported hereafter (Tab. 3.9).

Figure 3.5 a and b, show respectively the test modality and the local behaviours of each single layer during the test of two samples. Experimental observations led to note the influence of interlocking between external leaves and internal core on specimens. Furthermore, different stone typologies led to different failure modes.

Vintzileou and Miltiadou-Fezans (2008) carried out a laboratory campaign to study the typical strength of a three-leaf stone masonry with courses of solid bricks. The program was developed to study the effectiveness of ternary grouts (cement, pozzolan and hydrated lime) and hydraulic lime-based grouts on the compressive strength of masonry. These mixes were expressly designed to be applied in the case of panels with frescos, mosaics and decorative elements, with the aim to avoid any problem of durability and compatibility, as observed with high contents of cement. Obtained results, reported in Table 3.10, confirmed that the main failure mechanism in compression of this masonry typology is due to an early separation of outer layers. Nevertheless, both ternary and hydraulic lime-based grouts could induce a substantial enhancement of the masonry compressive strength. Finally, further analyses confirmed the capability of these grouts in filling the voids and homogenize the whole masonry.

Corradi et al. (2008) performed an experimental in-situ campaign on several panels of buildings to be dismantled. The preliminary phase involved compression tests on two panels. These were firstly tested in unstrengthened conditions and, subsequently, again subjected to compression after a combined intervention applying deep repointing and injection. The results are presented in Table 3.11.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Optimization of design for walls and pillars D4.1

Tab. 3.9 - Experimental results obtained by Binda et al. (2006).

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{wc}$ [N/mm²]</th>
<th>$E_{wc}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS3</td>
<td>5.8</td>
<td>1770</td>
</tr>
<tr>
<td>SS33</td>
<td>&gt;15.1</td>
<td>2940</td>
</tr>
<tr>
<td>NO3</td>
<td>6.4</td>
<td>2085</td>
</tr>
<tr>
<td>SO3</td>
<td>&gt;15.1</td>
<td>2725</td>
</tr>
</tbody>
</table>

Tab. 3.10 - Experimental results obtained by Vintzileou and Miltiadou-Fezans (2008).

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>$f_{w0}$ [N/mm²]</th>
<th>$f_{wc}$ [N/mm²]</th>
<th>$E_{w0}$ [N/mm²]</th>
<th>$E_{wc}$ [N/mm²]</th>
<th>$\varepsilon_{w0}$ [%]</th>
<th>$\varepsilon_{wc}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>three-leaf stone and solid bricks masonry</td>
<td>1.82</td>
<td>3.00</td>
<td>1000</td>
<td>1200</td>
<td>-</td>
<td>1.76</td>
</tr>
<tr>
<td>three-leaf stone and solid bricks masonry</td>
<td>1.74</td>
<td>3.75</td>
<td>1440</td>
<td>1550</td>
<td>1.60</td>
<td>2.50</td>
</tr>
<tr>
<td>three-leaf stone and solid bricks masonry</td>
<td>2.26</td>
<td>3.73</td>
<td>1500</td>
<td>1300</td>
<td>2.25</td>
<td>3.39</td>
</tr>
</tbody>
</table>

Fig. 3.5a - Masonry specimens with straight and keyed collar joints, shear and compression test set up.

(a) straight collar joints
(b) keyed collar joints

Fig. 3.5b - Stress-strain curve on different masonry layers (Binda et al. 2006).
Tab. 3.11 - Experimental results obtained from compression tests by Corradi et al. (2008).

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>Intervention</th>
<th>$\sigma_{\text{max}}$ [N/mm²]</th>
<th>$E$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double-leaf roughly cut</td>
<td>Unstrengthened condition</td>
<td>0.201</td>
<td>1289</td>
</tr>
<tr>
<td>stone masonry</td>
<td>Deep repointing + grout</td>
<td>0.286</td>
<td>4153</td>
</tr>
<tr>
<td>stone masonry</td>
<td>injections</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Double-leaf roughly cut</td>
<td>Unstrengthened condition</td>
<td>0.215</td>
<td>306</td>
</tr>
<tr>
<td>stone masonry</td>
<td>Deep repointing + grout</td>
<td>0.286</td>
<td>1770</td>
</tr>
<tr>
<td>stone masonry</td>
<td>injections</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Galasco et al. (2009a) realized a laboratory campaign on double-leaf stone masonry elements subjected to compression. This constitutes a preliminary phase to subsequent dynamic program involving shaking table tests on building models, realized with the same masonry typology. The aim was the mechanical characterization of the structural elements and, for this reason masonry piers (80x120x32cm) were realized. The compression tests were performed with subsequent cycles at increasing load levels. The final strength of panels is summarized in Tab. 3.12. Data show a limited scattering and values are aligned with other obtained during similar laboratory researches.

Tab. 3.12 - Experimental results obtained by Galasco et al. (2009a).

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{\text{sec}}$ [N/mm²]</th>
<th>$\varepsilon_{\text{sec}}$ [%]</th>
<th>$E_{\text{sec}}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>3.09</td>
<td>0.040</td>
<td>2400</td>
</tr>
<tr>
<td>V2</td>
<td>3.14</td>
<td>0.035</td>
<td>3000</td>
</tr>
<tr>
<td>V3</td>
<td>3.76</td>
<td>0.080</td>
<td>2900</td>
</tr>
<tr>
<td>V4</td>
<td>3.18</td>
<td>0.000</td>
<td>2200</td>
</tr>
<tr>
<td>V5</td>
<td>3.13</td>
<td>0.040</td>
<td>2900</td>
</tr>
<tr>
<td>V6</td>
<td>3.36</td>
<td>0.050</td>
<td>2200</td>
</tr>
</tbody>
</table>

Almeida (2010) performed uni-axial compression tests, Fig. 3.6, on three unreinforced stone masonry walls (PP1, PP2 and PP3) obtained from a 30.0m long wall, of a residence house in the center of Porto, which was to be demolished. This wall was cut into panels, forming approximately 1.2m large single walls with a height of 2.5m and a thickness of 0.4m. The walls were extracted and transported to the Laboratory of Earthquake and Seismic Engineering (LESE) in order to be tested under uni-axial compression. The walls’ masonry consisted of medium to large size stones (50.0 to 90.0cm measured diagonally) arranged in a regular alignment, including a significant number of small stone pieces (wedges), with mortar joints and, occasionally, brick pieces. The yellow granite stones have a rectangular type shape and exhibit reasonable good mechanical condition. The mortar joints showed variable thickness (0.5 to 2.0cm), are cream in colour and brittle.

Uni-axial compression tests, with and without lateral confinement of the walls, were performed to assess the unreinforced walls response, namely in terms of strength and stiffness, and to correlate the results with the survey and with the characteristics of the walls constituents: stone and mortar. In order to assess the influence of the walls internal voids on the panel stiffness, the wall PP3 was injected with a mortar made of a mixture of air hardening lime, hydraulic lime, sand and water with very poor stiffness and strength characteristics. The results, Figure 3.7, show that although the compressive strength is within the expected range of values, the stiffness is much lower than expected, especially when compared to code requirements and references in the literature. The injection of mortar to fill in the voids of PP3 showed that the observed large deformability is strongly related with these large voids, because, although it had very poor mechanical characteristics, the injected mortar led to a significant stiffness and strength increase of the wall, Figure 3.7b.
3.1.2 In-plane behaviour of masonry walls under combined vertical and horizontal loads

Tomaževic (1992) performed also an experimental campaign aiming at the shear resistance of the multi-leaf stone masonry. These were executed on masonry panels similar to those previously presented and tested under monotonic compression. In this case, experiments were performed on samples having a slenderness ratio equal to 2. Furthermore, both in-situ and laboratory tests were realized, Figure 3.8. Results are reported in Table 3.13.

A further experimental campaign was carried out by Tomaževic and Apih (1993) on two-leaf masonry walls, made of stones and crumbled bricks, with sub-horizontal mortar joints. Different cement-based grouts were employed with the insertion of pozzolana or hydrophobic additives. The execution details are similar to those of previous experiments performed by the author, namely, low pressure of injection (0.2-0.3bar) and regular mesh or holes (distance of 30÷50cm).
The program involved shear compression tests on 11 samples divided between both in-situ and laboratory experiments. The specimens tested in the ZRMK laboratories were subjected to a constant prestress of 1N/mm², considered about the 25÷30% of the masonry compressive strength after injection. Results, summarized in Table 3.14, demonstrate as the insertion of additives widely decreased the compressive strength of the grout without affecting the shear strength of the walls. Nevertheless, the addition of sand lowered the chemical incompatibility. The failure mechanisms highlighted as the injection allows the homogenization of the section and the improvement of the overall behaviour. This led to exploit the tensile strength of materials and to delay further failure mechanisms. Actually, all samples failed in shear at a lateral deformation of 3.0÷4.0mm and, close to the failure, vertical cracks occurred also on the transversal sides.

Modena and Bettio (1994) completed the experimental campaign of compression tests with further shearcompression tests employing a Sheppard testing system (dal Farra 1992). Masonry specimens were consolidated by the hydraulic lime-based grout injection, drilling about 10 holes per square meter on both sides of samples. Results showed as the intervention induced an increasing of both strength and stiffness. However, despite the great diffusion of holes, the panels were not uniformly injected and a poor adherence to the stones was noted. In the case of panels preven-
tively wetted these aspects could be improved as Table 3.15 shows.
The completion of the experimental campaign performed by Vintzileou and Tassios (1995), presented among the compression experiments, included also diagonal tests on two masonry panels without lateral interlocking (Fig. 3.9a) with overall dimensions of 80x80x40cm. Due to the brittle failure induced by this testing method, the specimens were tested only after injection. Both samples provided a tensile strength equal to 0.64 N/mm² (Fig. 3.9b). This confirms the capability of injection to widely increases the original mechanical characteristics, that can be assumed as: $f_{w,t,0} = 0.1 \cdot f_{w,c,0} = 0.15$ N/mm².

Beolchini et al. (1997a) carried out an experimental campaign of diagonal tests on stone masonry panels initially strengthened by cementitious grouts. After a first series of cyclic tests, the samples were repaired again using steel reinforcement and injections parallel to the masonry surface. A synthesis of results is presented in Table 3.16. Data demonstrate that a noticeable increase is induced in all considered mechanical parameters, such as strength, shear modulus and ductility ratio. This was particularly evident in the case of reinforced injections, which also showed a higher dissipation capacity close to collapse.

Subsequently, Beolchini et al. (1997b) designed also a further experimental campaign on some masonry panels of a building to be dismantled. Cyclic tests were performed imposing lateral displacements to the whole structure at floor levels, as presented in Figure 3.10. A first test was carried out up to the maximum lateral strength, without inducing the collapse the structure was subsequently repaired by means of widespread cementitious injections. The results demonstrated the capability to increase the overall original strength, (Fig. 3.11), of about 80%, in the case of lateral displacement applied only to the roof level, and of about 50% in the case of lateral displacement imposed at both floors levels.

### Tab. 3.15 - Experimental results obtained by Modena and Bettio (1994), [N/mm²].

<table>
<thead>
<tr>
<th>Panel</th>
<th>$\sigma_0$</th>
<th>$\tau_0$</th>
<th>$f_t$</th>
<th>$G$</th>
<th>$E$</th>
<th>$\sigma_0$</th>
<th>$\tau_0$</th>
<th>$f_t$</th>
<th>$G$</th>
<th>$E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.048</td>
<td>0.043</td>
<td>0.040</td>
<td>43.1</td>
<td>258.9</td>
<td>0.061</td>
<td>0.055</td>
<td>0.052</td>
<td>56.1</td>
<td>339.6</td>
</tr>
<tr>
<td>2</td>
<td>0.064</td>
<td>0.046</td>
<td>0.043</td>
<td>37.8</td>
<td>236.5</td>
<td>0.044</td>
<td>0.049</td>
<td>0.037</td>
<td>39.9</td>
<td>239.5</td>
</tr>
<tr>
<td>3</td>
<td>0.025</td>
<td>0.026</td>
<td>0.034</td>
<td>28.0</td>
<td>168.3</td>
<td>0.036</td>
<td>0.034</td>
<td>0.035</td>
<td>37.8</td>
<td>239.5</td>
</tr>
<tr>
<td>4</td>
<td>0.028</td>
<td>0.020</td>
<td>0.026</td>
<td>28.0</td>
<td>168.3</td>
<td>0.042</td>
<td>0.042</td>
<td>0.039</td>
<td>41.0</td>
<td>246.0</td>
</tr>
<tr>
<td>5</td>
<td>0.030</td>
<td>0.031</td>
<td>0.026</td>
<td>30.2</td>
<td>151.2</td>
<td>0.048</td>
<td>0.040</td>
<td>0.044</td>
<td>44.4</td>
<td>276.3</td>
</tr>
<tr>
<td>6</td>
<td>0.030</td>
<td>0.031</td>
<td>0.026</td>
<td>30.2</td>
<td>151.2</td>
<td>0.048</td>
<td>0.040</td>
<td>0.044</td>
<td>44.4</td>
<td>276.3</td>
</tr>
</tbody>
</table>

![Fig. 3.9 - Specimens tested by Vintzileou and Tassios (1995).](attachment:image.png)

(a) Diagonal test setup  
(b) $f_t - \varepsilon$ relationship
Tab. 3.16 - Experimental results obtained by Beolchini et al. (1997a).

<table>
<thead>
<tr>
<th>Test</th>
<th>Intervention</th>
<th>$\tau_o$ [N/mm$^2$]</th>
<th>$G$ [N/mm$^2$]</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>injection</td>
<td>0.222</td>
<td>111.15</td>
<td>5.55</td>
</tr>
<tr>
<td></td>
<td>r. injection (4 bars)</td>
<td>0.297</td>
<td>66.05</td>
<td>4.7</td>
</tr>
<tr>
<td>2</td>
<td>unstrengthened</td>
<td>0.066</td>
<td>8.2</td>
<td>&gt;2.6</td>
</tr>
<tr>
<td>3</td>
<td>unstrengthened</td>
<td>0.057</td>
<td>12.6</td>
<td>&gt;1.9</td>
</tr>
<tr>
<td></td>
<td>r. injection (2 bars)</td>
<td>0.252</td>
<td>66.5</td>
<td>8.95</td>
</tr>
<tr>
<td>4</td>
<td>unstrengthened</td>
<td>0.056</td>
<td>7.6</td>
<td>&gt;2.8</td>
</tr>
<tr>
<td></td>
<td>r. injection (2 bars)</td>
<td>0.204</td>
<td>120.1</td>
<td>&gt;9.9</td>
</tr>
</tbody>
</table>

Within the experimental campaign proposed by Toumbakari and van Gemert (1997) also diagonal compression tests were carried out on three panels strengthened by grout injections, with the same characteristics presented for the compression tests. The specimens had overall dimensions equal to 80x80x40cm$^3$ and transversal connections between external layers could be also surveyed. Experimental observations underlined as cracks occurred because of sliding of mortar joints and only in a limited way on stones. As mentioned by Tomaževic and Apih (1993) yet, the results, summarized in Table 3.17, highlighted a substantial invariance of shear strength from the strength of the injected grout.
Vignoli completed the in-situ compression tests with a further experimental campaign involving experiments in the same location with the aim to study the influence of injections also in the shear behavior, (Modena 1999). Also in this case, as for compression tests, jacketing and injections were applied to the masonry panels. Two different experimental setups were considered, namely shear compression tests and diagonal tests. First method involved samples similar to those tested in compression, while for the second methodology a mean thickness of 43÷50cm was considered. Results, summarized in Figure 3.12 and Table 3.18 in (R=Repaired; T=Shear), demonstrated that the interventions reduced both the overall ductility ratio and the shear modulus.

![Figure 3.12 - Results of specimens tested by Vignoli, (Modena 1999).](image)

**Tab. 3.17 - Experimental results obtained by Toumbakari and van Gemert (1997).**

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{c'}$ [N/mm²]</th>
<th>$\tau_u$ [N/mm²]</th>
<th>$\delta_{\tau_u}$ %</th>
<th>$\sigma_{c'}$ [N/mm²]</th>
<th>$\varepsilon_{c'}$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.4</td>
<td>0.45</td>
<td>1.41</td>
<td>0.52</td>
<td>0.87</td>
</tr>
<tr>
<td>2</td>
<td>14.6</td>
<td>0.30</td>
<td>-</td>
<td>0.7</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>5.2</td>
<td>0.30</td>
<td>2.00</td>
<td>0.30</td>
<td>1.50</td>
</tr>
</tbody>
</table>

![Image of shear compression tests and diagonal tests](image)

**Tab. 3.18 - Experimental results obtained by Modena (1999).**

<table>
<thead>
<tr>
<th>Location</th>
<th>Intervention</th>
<th>$\tau_u$ [N/mm²]</th>
<th>$\mu$</th>
<th>$G$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pieve Fosciana (A)</td>
<td>unstrengthened</td>
<td>0.379</td>
<td>5.30</td>
<td>179</td>
</tr>
<tr>
<td>Pieve Fosciana (B)</td>
<td>unstrengthened</td>
<td>0.491</td>
<td>-</td>
<td>435</td>
</tr>
<tr>
<td>Pieve Fosciana (B)</td>
<td>jacketing</td>
<td>0.573</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Pieve Fosciana (B)</td>
<td>jacketing</td>
<td>0.664</td>
<td>-</td>
<td>274</td>
</tr>
<tr>
<td>Pieve Fosciana (B)</td>
<td>injection</td>
<td>0.337</td>
<td>2.34</td>
<td>268</td>
</tr>
<tr>
<td>Canova (G)</td>
<td>unstrengthened</td>
<td>0.114</td>
<td>5.74</td>
<td>102</td>
</tr>
<tr>
<td>Canova (G)</td>
<td>jacketing</td>
<td>0.397</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Canova (G)</td>
<td>jacketing</td>
<td>0.364</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Castello (F)</td>
<td>unstrengthened</td>
<td>0.072</td>
<td>-</td>
<td>36</td>
</tr>
<tr>
<td>Merizzo</td>
<td>unstrengthened</td>
<td>0.061</td>
<td>-</td>
<td>74</td>
</tr>
</tbody>
</table>
The experimental campaign proposed by Toumbakari (2002) was completed with further diagonal compression tests on samples similar to those tested under compression. Fig. 3.13. Results, presented in Tab. 3.19, highlight that the injection appeared most effective in increasing the tensile strength rather than the compressive one. Furthermore, this intervention allowed decreasing the horizontal deformations up to 70%. As a consequence, when damages occurred in the specimens, the crack developments resulted accelerated.

![Fig. 3.13 - Results of specimens tested by Toumbakari (2002).](image)

(a) Diagonal compression tests  
(b) Typical crack pattern

Tab. 3.19 - Experimental results obtained by Toumbakari (2002).

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f_{t,0}$ [N/mm²]</th>
<th>$f_{t,σ}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>BDC1</td>
<td>0.44</td>
<td>0.60</td>
</tr>
<tr>
<td>BDC2</td>
<td>0.34</td>
<td>0.73</td>
</tr>
<tr>
<td>BDC3</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td>SDC1</td>
<td>0.47</td>
<td>0.50</td>
</tr>
<tr>
<td>SDC2</td>
<td>0.34</td>
<td>0.68</td>
</tr>
<tr>
<td>SDC3</td>
<td>0.28</td>
<td>0.59</td>
</tr>
</tbody>
</table>

A comprehensive in-situ campaign was performed by Corradi et al. (2003) in several buildings which were to be dismantled in the regions struck by the Umbria-Marchigiano earthquake of 1997-1998. All selected panels were constituted by multi-leaf stone masonry, in some cases with brick courses at regular distance. No strengthening intervention was applied, since the aim of the research was the mechanical characterization of historical multi-leaf stone masonry structures. Two different tests were carried out: shear compression and diagonal compression. In the first case, specimens had a dimension of 90x180cm², while in the second one squared samples with 120cm per side were obtained. Before the initiation of the shear compression tests, three cycles of compression were performed up to 0.1, 0.2 and 0.3 N/mm² respectively. This allowed estimating the elastic modulus of masonry. After this preliminary phase, lateral displacements were imposed and the results are presented in Table 3.20.

Further investigations involved diagonal compression tests on similar masonry typologies, results are summarized in Table 3.21. The set of results highlight as the scattering of the obtained shear strength values is very low and the average is higher than the highest shear strength recommended by the Italian Standards. On the contrary, elastic properties and shear modulus are very scattered. Finally, the execution of both kinds of tests on the same masonry typology highlighted as these setups lead to significantly different results and rising the problem of which test is representative of the real masonry behaviour.
The investigation on shear strength about multi-leaf stone masonry was deepened by Corradi et al. (2008) through further in-situ tests. As for the previous campaign, also in this case both shear compression and diagonal tests were performed. Results of shear compression experiments reported in Table 3.22 show a great scatter but shear strength as well as shear modulus manifests a large increase.

Results of diagonal compression tests showed (Tab. 3.23), as the strengthening uniquely by the execution of deep re-pointing induced an increasing of shear stiffness, while the shear strength can be improved only using injections.

![Survey of the thickness of testes masonry panels, (Corradi et al. 2003).](image)

**Table 3.20 - Experimental results from shear compression tests, (Corradi et al. 2003).**

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>$E$ [N/mm²]</th>
<th>$\tau_s$ [N/mm²]</th>
<th>$\sigma_0$ [N/mm²]</th>
<th>$G$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80-120 cm</td>
<td>917</td>
<td>0.219</td>
<td>0.3</td>
<td>546</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80-120 cm</td>
<td>1814</td>
<td>0.225</td>
<td>0.3</td>
<td>450</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>471</td>
<td>0.172</td>
<td>0.3</td>
<td>216</td>
</tr>
</tbody>
</table>

**Table 3-21 - Experimental results from diagonal compression tests, (Corradi et al. 2003).**

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>$\tau_s$ [N/mm²]</th>
<th>$G_{1/3}$ [N/mm²]</th>
<th>$\gamma_{1/3} \times 10^{-3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80-120 cm</td>
<td>0.072</td>
<td>30</td>
<td>0.791</td>
</tr>
<tr>
<td>one leaf only bricks</td>
<td>0.069</td>
<td>131</td>
<td>0.136</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>0.047</td>
<td>19</td>
<td>0.824</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>0.072</td>
<td>25</td>
<td>0.942</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>0.068</td>
<td>60</td>
<td>0.370</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>0.053</td>
<td>26</td>
<td>0.642</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>0.059</td>
<td>37</td>
<td>0.533</td>
</tr>
</tbody>
</table>
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Tab. 3.22 - Experimental results from shear compression tests, (Corradi et al. 2008).

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>Intervention</th>
<th>$\tau_u$ [N/mm²]</th>
<th>$\sigma_t$ [N/mm²]</th>
<th>$G$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>Unstrengthened condition</td>
<td>0.083</td>
<td>0.147</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>Deep repointing + grout injections</td>
<td>0.412</td>
<td>0.272</td>
<td>281</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>Unstrengthened condition</td>
<td>0.080</td>
<td>0.184</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>Deep repointing + grout injections</td>
<td>0.196</td>
<td>0.268</td>
<td>196</td>
</tr>
</tbody>
</table>

Tab. 3.23 - Experimental results obtained from diagonal tests by Corradi et al. (2008).

<table>
<thead>
<tr>
<th>Masonry texture</th>
<th>Intervention</th>
<th>$\tau_u$ [N/mm²]</th>
<th>$G_{1/3}$ [N/mm²]</th>
<th>$\gamma_{1/3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>Unstrengthened condition</td>
<td>0.059</td>
<td>37</td>
<td>0.533</td>
</tr>
<tr>
<td></td>
<td>Deep repointing + grout injections</td>
<td>0.157</td>
<td>731</td>
<td>0.070</td>
</tr>
<tr>
<td>Double-leaf roughly cut stone masonry</td>
<td>Unstrengthened condition</td>
<td>0.045</td>
<td>80</td>
<td>0.190</td>
</tr>
<tr>
<td></td>
<td>Deep repointing</td>
<td>0.054</td>
<td>232</td>
<td>0.076</td>
</tr>
</tbody>
</table>

The experimental campaign of compression tests carried out by Vintzileou and Miltiadou-Fezans (2008) realized three diagonal compression experiments. These were realized to evaluate the effectiveness of ternary grouts in increasing of tensile and shear strengths of multi-leaf stone masonry panels.

Figure 3.15 shows that the injected panels could bear a mean tensile stress of about 0.22N/mm² in two cases and about 0.34N/mm² in the third panel, demonstrating its contribution to increase the overall strength of the masonry.

The experimental campaign developed by Galasco et al. (2009a) involved also diagonal compression tests on panels similar to those previously presented for the compression part. The tests were performed cyclically at increasing load levels up to the failure. In all cases the stress-deformation behaviour was linear up to quiet high loads, but results show a greater scatter than that obtained for compression tests. Results are reported in Table 3.24a.

A further part was designed and performed by Galasco et al. (2009b), completing the experimental program of quasi-static tests on double-leaf stone masonry. Five shear compression tests were realized on slender samples (250x125cm) and on squat specimens (250x250cm). They were subjected to two different pre-compression level equal to 0.2N/mm² and 0.5N/mm². These represent two realistic stress states normally present on historical buildings. The tests were designed to obtain the two most typical failure modes, namely shear and flexure, depending on the slenderness ratio and on the pre-compression level. Results, presented in Table 3.24b, are in accordance with those obtained from diagonal compression tests.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

3.2 Laboratory tests on earthen materials and walls

3.2.1 Introduction

In comparison to the recent advances in research on stone and brick masonry, knowledge on the material properties and failure mechanisms of earthen building construction is limited and scattered.

The death tolls brought about by recent catastrophes in developing countries where many inhabitants lived in earthen dwellings (Gujarat, India 2001 and Bam Iran 2003 Earthquakes; Tamil Nadu 2004 Tsunami) have brought about research campaigns aimed at improving earth construction in terms of strength, seismic resistance and speed of construction.

However, due to the difficulties in predicting the behaviour of earth as a construction material, some consider compressed earth blocks and compressed stabilised earth blocks to be a safer alternative to un-stabilised earthen building materials (Kotak 2007). Stabilised earth blocks include the addition of cement, and compressed earth blocks are built by modern presses. These construction materials do not have the same properties as historic earthen materials.

While dynamic tests of un-reinforced and reinforced models have been carried out to visually or qualitatively assess different reinforcement and retrofitting techniques (Tolles et al. 2000, Torrealva et al. 2009), no testing campaigns comparable to Tomaževic (1992) and Vintzileou and Tassios (1995) aiming at defining quantitative properties (ultimate base shear coefficients and structural response modification factors before and after strengthening) have yet been carried out to better understand the behaviour of earthen materials. When campaigns have been carried out, the focus on one of three materials (adobe, cob or rammed earth) offer neither comparative results for nor enough data to produce reliable non-linear models, whether homogeneous or non-homogeneous.
A summary of tests carried out so far (for un-stabilised, i.e. non-cement stabilised materials) are listed in Table 3.25 to Table 3.27.

Table 3.25 – Overview on tests performed on adobe masonry test specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>Element tested</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive</td>
<td>Masonry unit</td>
<td>Quagliarini and Lenci (2010)</td>
</tr>
<tr>
<td></td>
<td>Mortar cylinder</td>
<td>Roselund (1990)</td>
</tr>
<tr>
<td>Bond strength between units and mortar</td>
<td>Wallette (strictly not as wide wallette, as its width is that of 1 brick)</td>
<td>Heath et al. (2007)</td>
</tr>
<tr>
<td>Compression and shear tests</td>
<td>Masonry unit</td>
<td>Quagliarini et al. (2010)</td>
</tr>
<tr>
<td>Diagonal compression test</td>
<td>Masonry unit</td>
<td>Gurumo (1992)</td>
</tr>
<tr>
<td>Flexural strength</td>
<td>Wallette</td>
<td>Torrealva (2009)</td>
</tr>
</tbody>
</table>

Table 3.26 - Overview on tests performed on rammed earth walls

<table>
<thead>
<tr>
<th>Test</th>
<th>Element tested</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive</td>
<td>Prism (10 cm cube)</td>
<td>Vargas Neumann (1993)</td>
</tr>
<tr>
<td></td>
<td>Cylinders</td>
<td>Bui and Morel (2009)</td>
</tr>
<tr>
<td>Diagonal compression</td>
<td>Wall segment</td>
<td>Roselund (1990)</td>
</tr>
<tr>
<td></td>
<td>87 No. 60x60x15 cm³</td>
<td></td>
</tr>
<tr>
<td>Shear by diagonal compression</td>
<td>8 No. 200 x 200 x 20 cm walls</td>
<td>Vargas Neumann (1993)</td>
</tr>
</tbody>
</table>

Table 3.27 - Overview on tests performed on cob walls

<table>
<thead>
<tr>
<th>Test</th>
<th>Element tested</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, Shear Strength, Flexural Strength</td>
<td>Prism (15,20,30 cm³ cube)</td>
<td>Ziegert (2003)</td>
</tr>
</tbody>
</table>

3.2.2 Compression and flexural tests on materials and small wall elements

3.2.2.1 Adobe

The compressive strength of masonry units is the most common tested property of historic adobe, as samples of adobe are relatively simple to obtain (testing has been carried out by Ghannad et al. 2006, Sikka and Chaudhry 2009, amongst others), with compressive strength values varying between 0.6 and 0.9 MPa. In order to better understand the influence of straw and sand on adobe masonry units, Quagliarini and Lenci (2010) constructed bricks made out of clay with sand and fibres in varying quantities, tested in compression. Results are shown in Figure 3.16.
3.2.2.2 Mortar

Testing of earthen mortar has been carried out on cylindrical samples (Zine-dine, 2000), on cubic samples (Venu, 1993) and (Walker and Stace 1997) and on prismatic ones (Bei, 1996).

Azeredo et al. (2007) conducted extensive testing of the compressive strength of earthen mortar to study which testing procedures can be considered to be appropriate enough to obtain reliable results. Their study focused on a methodology that is also used for cement/sand mortars, where 16 x 4 x 4 cm³ samples are first tested in bending and the remaining pieces are then tested in compression. Strain measurement methods and platen restraint were also investigated, since these parameters have an influence on the determination of compressive strength of earth mortars.

Conclusions were that the manufacturing of prismatic or cylindrical samples having a height to diameter ratio of 2 resulted in satisfactory results, which are considered to be more reliable than those obtained by employing cubic samples, due to the usual presence of confinement when the latter are used. With respect to curing samples, constant temperature and relative humidity are mandatory since earthen mortars can have a great variance in mechanical properties due to different equilibrium moisture content. For further results, such as kneecap systems, platen restraint effects and strain measurement alternatives see Azeredo et al. (2007).

In 2009, Azeredo and Morel studied the flexural strength of mortars, which plays a role on unit-mortar bond, in that the mortar’s flexural strength must be greater than its bonding strength in order for mortar splitting to occur. Testing was carried out by using 3-point loading on 16 x 4 x 4 cm³ samples prisms, with procedure and apparatus in accordance to standard EN 1015-11 (2000).

3.2.2.3 Small adobe masonry elements

Quagliarini et al. (2010) studied the compression and shear strength of adobe masonry for the determination of its homogeneous properties. Small walls measuring 0.69 x 0.31 x 0.39 m³ consisted of three adobe bricks, with mortar joints. Shear tests with vertical compressive load were also conducted. Since only two walls were tested, results were 0.8-1.2 MPa at failure. Shear strength was
0.2-0.4 MPa. These results are consistent with strength values cited by Loaiza (2002), of 1.2 MPa for compressive strength and 0.25 MPa shear strength.

Gurumo (1992) tested out-of-plane flexural bond strength with a simple bond wrench, giving variable but extremely low bonds of around 0.05 MPa (Morris 2009). Walker (1999) also studied the bonding strength between mortar joints and blocks by using the bond wrench test, showing higher bond strength values of 0.10 MPa, probably due to the fact that compressed blocks with stabilised mortar were tested.

In addition to testing the flexural tensile strength of mortar samples, Azeredo and Morel (2009) carried out tests to evaluate the bond and flexural strength of adobe masonry (i.e. units and mortar composites) by using the experimental procedure shown in Figure 3.17. Masonry tested was made with units of different composition (with and without cement) and mortars of different composition (with and without cement). Results on non-stabilized materials revealed flexural strengths of 0.4 to 1.1 MPa. Material strength theory was then used to determine the bond strength, based on the assumption of beam continuity before cracking. Results were in the same order of magnitude as those obtained by Walker (1999), despite that the testing methodologies was different.

![Figure 3.17 - Determination of flexural strength of adobe masonry: testing apparatus (left) and diagram (right) of testing program conducted by Azeredo and Morel (2009).](image)

### 3.2.2.4 Rammed Earth

The characterisation of rammed earth material properties is by most studies carried out by means of tests on 10 cm cubes (Vargas Neumann, 1993) or on cylinders with a length of 20 cm and diameter of 10 cm (Hall and Djerbib, 2004, Burrough, 2008, Lilley and Robinson, 1995, Hamilton et al. 2006). Studies performed by Maniatidis and Walker 2008 and Bui et al. 2008 have shown, however, that there is a difference between the results obtained with 10 cm cube samples and those obtained with bigger samples.

In order to investigate possible anisotropy Bui and Morel (2009) conducted compressive strength tests on bigger samples, 40 x 40 x 65 cm³ and 20 x 20 x 40 cm³. Uniaxial compressive tests were carried out both in the vertical direction (perpendicular to rammed earth compression beds, sample size 40 x 40 x 65cm³) and horizontal direction (parallel to rammed earth compression beds, sample size 20 x 20 x 40cm³), and measured compressive strength, Young’s modulus and failure moduli for each of the two directions. The difference in magnitude between compressive strength in perpendicular and parallel direction amounted to not more than 10 %, and results are shown in Table 3.28.
3.2.2.5 Cob

Mechanical properties of cob were thoroughly researched by Ziegert (2003), who performed compression tests (with strain monitoring) on 15 cm, 20 cm and 30 cm cubic samples from historic cob walls. Ziegert also carried out flexural and shear strength tests on 60 x 15 x 15 cm³ and 45 x 15 x 15 cm³ samples (Tab. 3.29).

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td></td>
</tr>
<tr>
<td>150 mm³ cube sample</td>
<td>[N/mm²] 0,65-1,22</td>
</tr>
<tr>
<td>200 mm³ cube sample</td>
<td>[N/mm²] 0,63-1,30</td>
</tr>
<tr>
<td>300 mm³ cube sample</td>
<td>[N/mm²] 0,72-1,03</td>
</tr>
<tr>
<td>Young’s Modulus</td>
<td>[N/mm²] 178,3-334,9</td>
</tr>
<tr>
<td>Flexural Strength</td>
<td>[N/mm²] 0,22</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>[N/mm²] 0,26-0,27</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>[N/mm²] 19,5-34,5</td>
</tr>
<tr>
<td>Bond Strength</td>
<td>[g/cm²] 58,7-127,2</td>
</tr>
</tbody>
</table>

Figure 3.18 - Compressive strength testing apparatus and results (Ziegert 2003)

Compressive strength testing apparatus and typical results are shown in Figure 3.18. On-site testing on cob was found by Ziegert (2003) to be feasible by means of pendulum hammer tests, which show good correlation with compressive strength. The flexural strength was found to be 0,22 MPa, which implies that a ratio of 0,2 existed between flexural strength and compressive strength of the samples, in good correlation with values observed for rammed earth (0,19 to 0,22 MPa Dierks and Ziegert, 1999). The shear strength of cob was determined by means of an experimental setup normally employed for testing mortar and concrete (Fig. 3.19), which has also been used for
rammed earth testing (Dierks and Ziegert, 1999). The relation between shear strength \( (f_s) \) and compressive strength \( (f_c) \) was found to be \( f_s = 0.29 \) to \( 0.42 \cdot f_c \). A somewhat lower ratio was found to exist for rammed earth, with \( f_s = 0.27 \) to \( 0.33 \cdot f_c \) (Dierks and Ziegert, 1999). Results by Ziegert (2003) show that cob has a lower shear modulus than all other massive construction materials. The shear modulus is 0.09 to 0.12 times the elastic Modulus, whereas for rammed earth the shear modulus is 0.14 to 0.22 times the elastic Modulus.

![Experimental setup](image)

**Figure 3.19** - Experimental setup employed by Ziegert (2003) to determine shear strength and shear coefficient of cob (left), and results in the form of Stress-Strain diagram (right).

3.2.3 Compression and diagonal compression tests on wall elements

3.2.3.1 Adobe masonry

Within the frame of a program focusing on the strengthening of adobe houses, Hernandez et al. (1980) tested adobe wall specimens in simple compression, diagonal tension and flexure on both, the vertical and horizontal axes of the walls. The same tests were conducted in 1988 by Tolles and Krawinkler. Figure 3.20 represents the comparison between results from Hernandez (1980), referred to as “Prototype”, and Tolles and Krawinkler (1988), referred to as “Model”.

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Prototype</th>
<th>Stress (psi)</th>
<th>Stress (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression</td>
<td>Model</td>
<td>135</td>
<td>9.8</td>
</tr>
<tr>
<td>Flexure #1 (vertical)</td>
<td>Model</td>
<td>39.2</td>
<td>2.8</td>
</tr>
<tr>
<td>Flexure #2 (horizontal)</td>
<td>Model</td>
<td>20.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Tension</td>
<td>Model</td>
<td>19.0</td>
<td>1.3</td>
</tr>
</tbody>
</table>

(N.A. = data not available)

**Table 1: Comparisons of Prototype and Model Assembly Strength Properties**

![Fig. 3.20 – Test results on wall specimens](image)
3.2.3.2 Rammed Earth walls

A parametric study was carried out for rammed earth by Vargas Neumann (1993) with static diagonal compression tests on 0.60 x 0.60 x 0.15 m³ wall specimens and shear tests on 2 x 2 x 0.20 m³ walls. Varying parameters were soil granulometry, water content prior to compaction, level of compaction, use of additives, joint treatment. 87 diagonal compression tests were carried out, which showed that an increase in the amount of water in the mix prior to compaction increased the diagonal compressive strength, and that the level of compaction also increased strength up to a certain level, but that its influence on the strength was lower than that of the amount of water. Walls reinforced with cane were also tested.

Jaquin et al. (2006) constructed and tested a number of small test walls using a displacement controlled hydraulic press with a load cell – with a loading rate of approximately 3.6 N/min, a wall thickness of 300 mm and loads were applied over their full thickness with a timber beam of width 60mm. By comparing experimental results with analytical results obtained by means of FE modelling, he concluded that the layered nature of rammed earth must be taken into account when input data on mechanical properties of rammed earth from laboratory is taken into account (Fig. 3.21).

![Figure 3.21 – Results from Jaquin et al. (2006).](image)

3.2.3.3 Cob walls

So far no experimental mechanical data on larger cob wall segments are known to the authors.

3.2.4 In-plane behaviour of masonry walls subjected to shear loading

3.2.4.1 Adobe masonry

Gulkan and Gurdil (1989) investigated the behaviour of 1 x 1 x 0.3 m³ square adobe wall panels subjected to constant in plane compression normal and horizontal to the bed joints and to incrementally applied diagonal load for compressive and shear forces. This determined that failure of wall specimens under combined compressive and diagonal loads occurred at joint separation for low magnitude compressive loads and crushing or splitting for higher compressive loads. The experimental setup of this test is shown in Figure 3.22.

![Figure 3.22 – Experimental setup of adobe wall panel test.](image)
In 1992, Gurumo conducted diagonal compression tests on 1.2 m adobe panels with different reinforcement. Results indicated that reinforced samples carried almost twice the load of un-reinforced adobe but experiments were carried out with stabilized adobes which are not part of the NIKER project. Sathiparan et al. (2006) conducted diagonal compression tests on non-reinforced and PP reinforced adobe masonry wall specimens. These tests were conducted on stabilised masonry walls, the results of which are not comparable to those of historic adobe masonry. San Bartolome et al. (2009) tested four small walls with 0.80 m width under diagonal compression to evaluate the shear resistance. Tests were not effective as detachment occurred during the handling prior to the test, resulting in very low values for shear resistance, which are not cited in the publication.

Figure 3.22 - Experimental setup adopted by Gulkan and Gurdil (1989).

3.2.4.2 Rammed earth walls
In addition to the parametric study based on simple compression tests carried out by Vargas Neu- mann (1993), eight shear tests on 2 x 2 x 0.20 m³ walls were carried out in shear loading. The conclusion of the testing program was that the clay, water content and compaction are the dominating influences on the shear resistance of rammed earth. His findings indicated that clayey soils with a moisture content of 20 % higher than determined by the standard proctor test and an optimized compaction yielded the best results. Results showed that rammed earth walls were more resistant to earthquakes than adobe masonry walls by 40%.

3.2.4.3 Cob walls
No test results on combined vertical and horizontal loads of cob walls are known to the authors so far.

3.2.5 Cyclic testing
3.2.5.1 Adobe
Gibu et al. (1993) conducted a series of in-plane cyclic loading tests on 8 types of confined masonry walls with and without different types of confining elements (beams-columns at the edge, smalls columns distributed, orthogonal walls, effect of slab) and amount of reinforcement (longitudinal and transversal) in the confining elements. Figure 3.23 shows the hysteretic curve and crack pattern of one specimen tested. It is not clear whether dead load of the roof was accounted for during testing.
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Grant Agreement n° 244123

Figure 3.23 - Hysteretic curve (left) and final state (right) results of in-plane cyclic loading on adobe walls (Gibu et al. 1993)

Morris and Walker (2000), using a test layout similar to that shown in Figure 3.24 (left), tested several 1.2 x 1.8 m² adobe walls in-plane. Figure 3.24 (right) shows how slipping in the mortar planes gave effective ductility in a wall with vertical reinforcement.

Figure 3.24 - Apparatus (left) and hysteretic curve (right) of adobe wall tested by Morris (1993)

Torrealva (2009) conducted in-plane shear testing on wall elements consisting of 3 interconnecting walls, both un-reinforced and reinforced with geotextiles or geotextiles covered in plaster. It is not clear from the tests whether the wall was subjected to vertical loading equivalent to that resulting from a roof structure. A comparison between the three types of walls (non retrofitted, retrofitted with geotextiles and retrofitted with geotextiles and plaster is shown in Figure 3.25).

Figure 3.25 - Hysteretic curve for non-retrofitted (left), retrofitted with geotextiles (middle) and retro-fitted with geotextiles and plaster (right) walls is (Torrealva 2009)

3.2.5.2 Rammed earth
Several rammed earth walls were tested under in-plane cyclic loading by Walker and Morris (1998), the hysteretic behaviour of which is shown in Figure 3.26. These were however earth panels stabilised with 10% cement.

![Hysteretic curve of 2.4m high and 1.8m wide wall reinforced vertically at each end and horizontally loaded on top (Walker and Morris 1998).](image-url)

Hamilton et al. (2006) conducted in plane (4 specimens) and out of plane cyclic testing on 3% cement stabilised rammed earth walls with and without post tensioned reinforcement. The walls were constructed with concrete caps and bases.

### 3.2.6 Dynamic testing

Extensive dynamic testing has been carried out on adobe 1:5 and 1:2 models. The first shaking-table tests were carried out in Mexico during the 1970s (Hernandez, Meli and Padilla 1980), when five 1:2.5 model adobe buildings were tested with the aim of determining the effectiveness of concrete bond beams, horizontal steel rods, and welded wire as reinforcement or retrofitting techniques.

Early research on the strengthening of adobe construction by means of internal cane reinforcement and wooden bond beams was carried out in the 1980s at the Catholic University in Lima, Peru, where material properties, tilt-table tests (Vargas-Neumann and Otazzi 1981), shaking-table tests (Vargas-Neumann, Bariola, and Blondet 1984), and dynamic tests on the out-of-plane stability of adobe walls were performed (Bariola Bernales 1986). Also in the 1980s, dynamic tests were conducted at Stanford University, Palo Alto, California on 1:5 models in order to evaluate the effects of simple retrofitting techniques on their dynamic behaviour (Tolles and Kra-winkler 1989). Large-scale model tests were then conducted at UC Berkeley, California, on adobe models fitted with wooden bond beams and steel wire mesh (Scawthorn and Becker 1986).

Shake table testing aiming at improving the seismic resistance of adobe structures was carried out further in Peru (Bariola et al. 1989; Zegarra et al. 1999; Quiun et al. 2005), Mexico (Flores et al. 2001), the United States (Tolles et al. 2000) and Colombia (Yamin et al. 2004). Shake table testing was also the basis on which the Getty Conservation Institute initiated an assessment of appropriate retrofitting techniques for historic structures (Tolles et al. 2002). The most recent advance in retrofitting is described by Torrealva (2009), who performed shake table testing on structures retrofitted with geotextile materials. Information on shake table testing at Bristol University is not yet available in the literature, but is available through the following link:

[http://www.historicrammedearth.co.uk/earthquake.htm](http://www.historicrammedearth.co.uk/earthquake.htm)
4 EXPERIMENTAL METHODS WITHIN THE WORKPACKAGE

4.1 General approach

The approach for experiments on vertical elements within WP4.1 needs to accomplish two tasks. One task will consist in the general assessment of parameters for walls and pillars of different material combinations. As it was shown in the previous chapter 2.1, the combination of different materials with different construction methods generates structural elements with a vast variety of properties. Unfortunately knowing crucial parameters of the single materials, i.e. mortar and stone, does not necessarily allow the calculation of an entire structural element. It is therefore necessary to experimentally determine the parameters for wall segments built with the material combinations and type of construction method in question. The results of the tests will yield the input parameters for the design optimization, which will be performed by numerical modelling in WP4.2. The experiments themselves will be realized by static loading experiments where load-bearing behaviour and displacement are measured in various directions and where the crack pattern is being analyzed.

The second task of the experiments includes the in-depth analysis of the proposed strengthening techniques. This task will be performed on specimens and tests, similar to the previous task, on 1:1.5 wall elements in dynamic in-plane loading and in form of in-situ tests of strengthened historical masonry. Once again, the results of these tests will be used for the calibration of the models used in the numerical calculation of the response of historical walls towards seismic load.

4.2 Materials and wall types

Table 4.1 lists the type of materials and wall types used in WP4.1. Even though this choice of materials gives only a small range out of the vast number of material varieties but the selections fall each within a group of the principle building materials used in historic wall constructions: earth, brick and stone.

Tab. 4.1 - Materials and wall types of WP4.1.

<table>
<thead>
<tr>
<th>Wall type</th>
<th>Block materials</th>
<th>Mortars</th>
<th>Specifics masonry type</th>
<th>Specifics materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry walls</td>
<td>Limestone</td>
<td>Hydraulic lime High lime</td>
<td>Three leaf masonry Two leaf masonry</td>
<td>Limestone:</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- medium-big elements (&gt;15cm, &lt;25cm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mortar: - Hydraulic Lime, with a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>low content of water-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>soluble salts and with a</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>controlled shrinkage</td>
</tr>
<tr>
<td>Brick</td>
<td>Hydraulic lime</td>
<td></td>
<td>One leaf masonry</td>
<td></td>
</tr>
<tr>
<td>Adobe</td>
<td>Earth</td>
<td></td>
<td>One leaf</td>
<td>Uncompressed handmade adobe, low fiber content</td>
</tr>
<tr>
<td>Masonry pillars</td>
<td></td>
<td></td>
<td></td>
<td>earth mortar</td>
</tr>
<tr>
<td>Monolithic –</td>
<td>–</td>
<td>–</td>
<td>One leaf</td>
<td>Compressed with low water content, no fibers</td>
</tr>
<tr>
<td>Rammed earth</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monolithic –</td>
<td>–</td>
<td>–</td>
<td>One leaf</td>
<td>Uncompressed with high</td>
</tr>
<tr>
<td>Cob</td>
<td></td>
<td></td>
<td></td>
<td>fiber content</td>
</tr>
</tbody>
</table>

Except for the in-situ experiments the materials used in the laboratory experiments will not consist of original materials. However, their properties will be similar and as close as possible to historical materials utilized in the past. In any case, the properties of the new materials are already known or will be determined in the course of WP4.1.
4.3 Specimen type and type of loading

The experiments will be performed on two different levels:

1. On the material level with the single masonry components.
   - Stone
     - Determination of the flexural strength, (three point bending, EN 12372)
     - Determination of the static elastic modulus (EN 14580)
     - Determination of the real and apparent density, and of the total and open porosity (EN 1936)
     - Determination of the uniaxial compressive strength (EN 1926)
   - Mortar
     - Determination of flexural and compressive strength of hardened mortar (EN 1015-11)
     - Modulus of elasticity
     - Shrinkage
   - Injection Materials
     - Determination of the flexural and compressive strength of hardened mortar (EN 1015-11)
     - Injectability tests on representative cylinders of the walls inner layer and its mechanical characterization (to be evaluated before the beginning of the experimental phase)
     - Rheology
     - Setting
     - Adhesion strength
     - Shrinkage
     - Modulus of elasticity
     - Durability experiments in form of freeze/thaw and salt crystallization tests
   - Clay Bricks characterization through mechanical tests (UNIPD)
     - Three-point bending
     - Compression tests
     - Splitting tensile tests
   - Earth blocks, rammed earth, cob
     - Compressive strength
     - Modulus of elasticity
     - Apparent density and porosity
     - Mineralogical composition
     - Particle size distribution

2. On the structural level with masonry/wall specimens.
   - Static tests
     - Compression tests
     - Combined shear and compression tests
     - In-situ: Diagonal tests
     - In-situ: Pull-off low-intrusive tests (applied to rammed earth, adobe or cob walls) with Helifix pull-off equipment (to obtain correlation with compression load tests on in-situ samples).
   - Dynamic tests
     - In-plane cyclic shear loading
     - Horizontal unidirectional in-plane loading
   - Non destructive tests (NDT) and Semi destructive tests (SDT)
     - Non destructive tests on the In-situ and Laboratory stone masonry specimens
     - Dynamic identification tests on the laboratory stone masonry specimens
     - Flat-Jack Tests

The tests on the masonry/wall specimen will be performed on two different types of samples:
   - Smaller walls in the scale of 1:3 to 2:3
• Real scale or near real scale walls 1:1 to 1:1.3

Another set of cyclic tests will be performed on adobe masonry and rammed earth walls. Here the moisture content of the material will be increased. Since earthen materials are in particular responsive to changes in moisture content this will help to understand the role of degradation effects and higher moisture contents during earthquakes.

To ensure a proper use of composite materials for structural strengthening of clay brick structures, deeper investigations on local phenomena are still in need, since they strongly affect the design and the effectiveness of the intervention. As so, experimental tests will be performed studying the local behaviour and the durability of composites, in particular CFRP and GFRP, applied on clay brick substrates.

1) Assess the quality of the adherence of the reinforcement to the clay elements through:
   - Pull off tests
   - Shear tests

2) Assess the durability of the reinforcements through tests simulating environmental effects such as salt crystallization, thermal cycles and moisture.

4.4 Strengthening techniques

Strengthening techniques will be applied to real scale or smaller walls in multiple ways. For crack repair, injections grouts developed in WP3.2 will be applied in order to reestablish force flow and structural stability. The grouts are based on hydraulic lime and are used for stone and adobe masonry as well as for rammed earth and cob walls. For strengthening vertical elements various strategies will be developed:

• Grouting with hydraulic lime (including natural hydraulic lime). This will be applied to stone masonry walls specimens, earth block, rammed earth and cob specimen tested as part of the laboratory experimental campaign performed by the UNIPD.
• Grouting with an hydraulic lime mixture. The technique will be applied to the stone masonry walls tested in-situ (Abruzzo, Italy) by the UNIPD. The quantity of grouting is going to be measured in order to attain an estimation of the void percentage on the chosen walls.
• Application of composite materials (CFRP and GFRP) for the strengthening of brick masonry structural elements.
• Structural repointing: Performed with stainless steel bars and composite materials in conjunction with hydraulic lime mortar for repointing. The technique will be applied to brick and stone masonry.
• Intra and extra mural anchoring with stainless steel rods and bars made from glass fiber reinforced polymers (GRP). The method will be utilized on adobe masonry, rammed earth and cob walls.
• Surface mounted anchored plastic nets. This will be utilized on rammed earth walls.
  - Spatial stabilization of clay
  - Maximum compaction based on the addition of a new stabilizing agents:
    - Stabilizing binders (Portland cement, lime air)
    - Stabilization LTGS (Low temperature-measuring geopoly petting)
• Reinforcement, consisting in adding the cohesion of fibrous material that provides a good overall consistency by means of frictional forces.
• Glass fiber and epoxy resin composite, with confinement connectors (steel anchors or CIN-tec anchors).
• Polypropylene net, covering both wall surfaces along with confinement connectors (transversal) or superficial fixing devices (RICHTERGARD system).
• Earth reconstruction and/or external reinforcement, using a projection technique (applied to rammed earth damaged vertical walls, when traditional methods cannot be performed in existing constructions).
On some of the repair techniques durability and compatibility tests will be performed. This concerns mainly grouting materials and FRPs with the focus on their long term stability.
5 DETAILED EXPERIMENTAL WORK PLAN FOR EACH PARTNER

5.1 Partner n° 1 – UNIPD

5.1.1 Stone masonry walls - Laboratory experimental campaign

5.1.1.1 Introduction and Objectives

The contribution from the Department of Structural and Transportation Engineering of the University of Padua includes a series of static tests on stone masonry walls. The study considers 3 leaves stone masonry walls with and without strengthening. The consolidation technique considered on this study is injection with mortar provided by Tassulo. The used masonry was projected and created according to the original constructive technique representative of the existing typology on historical buildings throughout Europe.

The static tests are composed essentially by simple compression test and shear-compression in-plane cyclic tests on reinforced and unreinforced stone masonry panels which allow characterizing the complex mechanical behaviour of this material and elements.

These tests are complemented also by non destructive testing (NDT) and semi destructive testing (SDT) (mainly sonic tests and flat jacks) and by laboratory tests on the material itself (masonry), on its composing elements (stone and natural lime mortar) and on the injection material in order to more accurately characterize mechanically, physically and chemically the applied material.

The main objective of this contribution is to gather a large amount of information on the static and dynamic behaviour of stone masonry and stone masonry elements (under reinforced and unreinforced conditions), in order to accurately characterize its mechanical behavior and determine constitutive laws, extremely useful for the numerical simulation of this type of material and structure.

5.1.1.2 Experimental Program

The research includes a series of static mechanical tests performed at the Building Materials Experimental Laboratory (UNIPD) on 3 leaves stone masonry panels in 1:1 and 2:3 scale and 1 leave stone masonry walls in scale 1:1, under original conditions and with strengthening injections. The mechanical characterization of the material and assemblage will be integrated on the NDT (Sonics) and SDT (Flat-jacks).

The experimental program will include the realization of:

- 6 panels in scale 1:1 for simple compression tests (0.5x1.0x1.2m³ and a 20.0cm reinforced concrete (RC) beam on top and bottom) from which 3 of them will be tested under strengthened conditions;
- 8 panels in scale 1:1 for in-plane shear-compression tests (0.5x1.0x1.2m³ and a 30.0cm RC beam on top and a 25.0cm RC beam on bottom) from which 4 of them will be tested under strengthened conditions;
- 6 panels in scale 2:3 for simple compression tests (0.33x0.80x1.0m³ and a 20.0cm RC beam on top and bottom) from which 3 of them will be tested under strengthened conditions;
- 8 panels in scale 2:3 for in-plane shear-compression tests (0.33x0.80x1.0m³ and a 30.0cm RC beam on top and a 25.0cm RC beam on bottom) from which 4 of them will be tested under strengthened conditions;
- 6 panels in scale 1:1 (0.5x1.5x2.2m and a 20.0cm RC beam on top and bottom) from which 3 of the them, under strengthened conditions, will be tested using flat-jacks (single and double);
• 4 stone masonry panels of 1 leave in scale 1:1 for simple compression tests (0.18x1.0x1.2m and a 20.0cm RC beam on top and bottom).

According to the experience attained in previous experimental works, it is scheduled to be constructed 6 series of continuum walls with top and bottom confining RC beams, which will be divided into panels (Fig 5.1). This choice will allow attaining a clean cut surface of the wall section, allowing afterwards an easier and significant survey of the walls details, to estimate with great accuracy the percentage of composing elements (stone, mortar and void). In addition, for practical reasons, in the construction of the walls the terminal faces on each side should be provided with a minimum overlapping and bonding connection; the continuum construction will allow to avoid this connection throughout the panels sections and to allow the homogenisation of the constructive typology of the panels. An intermediate portion "to be lost" will separate the walls to be left in the original conditions from the ones to be injected with strengthening material. The injections are going to be made before the walls are cut into single panels, and so, the intermediate borders will be isolated in order for that the injections won’t spread to the panels.

The panels will be constructed at the Scuola Edile di Verona (Verona, Italy) and then subdivided and transported to the Building Materials Experimental Laboratory at the UNIPD, (Padua, Italy), for testing.

In a preliminary phase the identification of the base properties of the masonry composing materials (stone and mortar) and of the injection mixture is scheduled by the following tests:

**Stone**

- Determination of the flexural strength under concentrated load, (three point bending - EN 12372);
- Determination of the static elastic modulus, (EN 14580);
- Determination of the real and apparent density, and of the total and open porosity, (EN 1936);
- Determination of the uniaxial compressive strength, (EN 1926).

**Mortar**

- Determination of the flexural and compressive strength of hardened mortar, (EN 1015-11).

**Injection Material**

- Determination of the flexural and compressive strength of hardened mortar, (EN 1015-11).
- Injectability tests on representative cylinders of the walls inner layer and its mechanical characterization (to be evaluated before the beginning of the experimental phase).
Fig. 5.1 - General description of the continuum walls.
5.1.2 Stone Masonry Walls - In-situ Experimental Campaign

5.1.2.1 Introduction and Objectives

The contribution from the Department of Structural and Transportation Engineering of the University of Padua includes also a series of in-situ diagonal tests on two leaves stone masonry walls typical for the region of Abruzzo (Italy). These walls (test specimens) were chosen among the damaged buildings on the localities of Onna, S. Eusanio Forconese and Tempera (Abruzzo, Italy).

These tests will be complemented also by non destructive test (NDT), in particular sonic tests and by laboratory tests on the material itself (masonry), on the injection material in order to more accurately characterize mechanically (compression and flexural strength) the applied material.

5.1.2.2 Experimental Program

Basically, the experimental in-situ campaign will consist of:

1. Finding safe and suitable walls to perform the experimental tests;
2. Perform preliminary sonic velocity characterization of the unreinforced walls with sonic tomography;
3. Injection of the walls with a hydraulic lime mixture up to half of its length, measuring the quantity of grout in order to attain an estimation of the void percentage on the chosen walls;
4. Post-injection sonic characterization of specimens;
5. Cut the walls in order to obtain the isolated specimens, indicatively 2 non-injected and 2 injected specimens (0.90 m x 0.90 m and thickness varying between 0.50 m and 0.60 m);
6. Laboratory tests to characterize the compressive and flexural strength of the mixture sampled during the injection and matured in controlled and environmental conditions;
7. Perform monotonic diagonal in-situ tests on the reinforced and unreinforced walls.

Fig. 5.2 - In-situ diagonal experimental test setup.
5.1.3 Experimental campaign on composite materials

5.1.3.1 Introduction and objectives
The application of composite materials may be presently considered as a viable solution for strengthening of brick masonry structural elements. To ensure a proper use of composite materials for structural strengthening, deeper investigations on local phenomena are still in need, since they strongly affect the design and the effectiveness of the intervention.

Three experimental campaigns have been designed to study the local behaviour and the durability of composite materials applied on masonry substrates. The first issue will be analyzed through Pull-off test series, by forcing the interface to actions normal to its surface, and through Shear Tests, by applying actions tangential to the reinforcement axis, while durability will be investigated through tests simulating environmental effects such as salt crystallization, thermal cycles and moisture.

Detailed results of the hereafter described experimental activities will be reported within the Deliverable 3.7 and/or Deliverable 4.3.

5.1.3.2 Experimental program on Pull-off Tests
This activity (see Tab. 5.1) is based on performing three types of mechanical tests on the same clay element, namely a three-point bending on the intact brick, a compressive test or a splitting tensile test on one of the two portions produced by the flexural failure, and two pull-off tests on the remaining clay piece, after the application of a layer of FRP textiles on both wider surfaces. Two types of solid clay bricks will be used: extruded and facing ones produced by different manufacturers; carbon fibers (CFRP) and glass fibers (GFRP) will be applied by means of an epoxy matrix as reinforcement. Based on the results, the possible relations between the pull-off strength and each of the other parameters (flexural, compressive and splitting tensile strength) will be investigated, taking into account the variation of the clay substrate, the type of fibres and the presence, or not, of a layer of primer. The research has already started and part of the tests have been performed: the program will be completed during the present Project, and the final analyses will be developed.

Tab. 5.1 - Text matrix for Pull-off Tests

<table>
<thead>
<tr>
<th>Series</th>
<th>Brick Type</th>
<th>Flexural tests</th>
<th>Compressive tests</th>
<th>Splitting tests</th>
<th>Pull-off tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>extruded</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP w/o primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>GFRP with primer</td>
</tr>
<tr>
<td>S2</td>
<td>facing</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>GFRP with primer</td>
</tr>
<tr>
<td>S3</td>
<td>extruded</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>CFRP w/o primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>GFRP with primer</td>
</tr>
<tr>
<td>S4</td>
<td>facing</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>GFRP with primer</td>
</tr>
<tr>
<td>S5</td>
<td>extruded</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
<tr>
<td>S6</td>
<td>extruded</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>CFRP with primer</td>
</tr>
</tbody>
</table>
5.1.4 Experimental program on shear tests

Since few contributions are available concerning debonding problems on masonry, an extensive campaign (see Tab. 5.2) focused on the local shear behaviour of externally bonded FRP, applied on one type of solid clay brick, have been planned. Four types of reinforcements will be tested, namely carbon (CFRP), glass (GFRP), basalt (BFRP) and steel (SRP) fibres. Two different set-ups, Single-lap and Double-lap, will be compared.

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Type of reinforcement (number of samples)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single lap</td>
<td>CFRP 3</td>
</tr>
<tr>
<td>Double-lap</td>
<td>CFRP 3</td>
</tr>
</tbody>
</table>

5.1.5 Experimental program on durability (in cooperation with POLIMI)

The activity is focused on investigating the durability of masonry repaired by composite materials. FRP strips are applied on masonry elements (single bricks or small assemblages with mortar joints) and tested in aggressive environment conditions as in presence of salts or to evaluate the influence of moisture and temperature on the FRP-masonry bond.

This experimental campaign (see Tab. 5.3) includes the following tests: (1) crystallisation tests on masonry assemblages, (2) Pull-off tests after thermal cycles and (3) Pull-off tests in different masonry moisture conditions before FRP application: environmental, saturated and semi-saturated.

Aim of the research is the analysis of several parameters that could affect the bond adhesion in time, as well as in "real" environmental conditions. Specimens are single solid bricks or small assemblages simulating diffused masonry textures such as: (a) shiner course specimens, with courses of bricks laid with the long sides upright (as in thin vaults), (b) stretcher course specimens simulating common textures of masonry walls.

Salt crystallization tests will be performed according to RILEM TC 127-MS. Concerning thermal ageing and adhesion/absorption tests, appropriate procedures will be defined and optimized during the Project.

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Specimen type</th>
<th>Number of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</table>
5.2 Partners n° 2 and 17– BAM in collaboration with ZRS: Tests on earthen structural elements

5.2.1 Scope and objectives

The contribution of BAM and ZRS to this work package is focused on the determination of general material parameters and the behaviour of wall and wall test specimens made from earth towards different loading scenarios. In a second step, strengthening techniques will be investigated on different types of earthen wall elements in form of static and cyclic loading experiments. The experimental program is entirely performed in the laboratories of BAM. The types of wall segments considered in the experiments consist of one leaf masonry of adobe in combination with earth mortar and monolithic walls in form of rammed earth and cob specimens (Tab. 4.1). The investigations will be carried out on several levels in order to acquire the mechanical behaviour of the single materials as well as the structural elements (wall segments) as elaborated in Chapter 4.

The goal of the experiments is to acquire a basic knowledge of the mechanical properties of different structural elements and the criteria of performance of wall segments under dynamic loading. Performance criteria of the latter topic include moisture content. The mechanical properties will be used as input values for numerical modelling in WP4.2. Strengthening techniques will be tested under static and dynamic loading and consist of grouting and the utilization of anchors consisting of stainless steel and/or fibre reinforced polymer bars.

5.2.2 Materials and material properties

The earthen materials for the experiments are from a local manufacturer of earthen building products. The grouting mortar is being developed within work package 3.2 and consists of a lime based material with pozzolanic additions. Anchors will be selected according to the specifications concerning the wall type and accelerations on the sliding table. The materials for the different wall constructions are listed in Table 5.4.

<table>
<thead>
<tr>
<th>Tab. 5.4 - Materials for wall constructions and reinforcement.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of application</strong></td>
</tr>
<tr>
<td>Masonry segments</td>
</tr>
</tbody>
</table>
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

<table>
<thead>
<tr>
<th>Masonry composing material</th>
<th>Property</th>
<th>Standard, test procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adobe blocks</td>
<td>Compressive strength</td>
<td>DIN V 105-1</td>
</tr>
<tr>
<td>Rammed earth</td>
<td>Modulus of elasticity</td>
<td>DIN 1048-5, modified</td>
</tr>
<tr>
<td>Cob</td>
<td>Apparent density</td>
<td>DIN V 105-1</td>
</tr>
<tr>
<td></td>
<td>Particle size</td>
<td>Sieve and sedimentation analysis DIN 18123</td>
</tr>
<tr>
<td>Earth mortar</td>
<td>Particle size</td>
<td>Laser granulometer</td>
</tr>
<tr>
<td></td>
<td>Shrinkage</td>
<td>Strain gages</td>
</tr>
<tr>
<td></td>
<td>Compressive strength</td>
<td>DIN EN 1015-11</td>
</tr>
<tr>
<td></td>
<td>Flexural strength</td>
<td>DIN EN 1015-10</td>
</tr>
<tr>
<td>Injection material</td>
<td>Apparent density</td>
<td>DIN EN 1015-10</td>
</tr>
<tr>
<td></td>
<td>Adhesion strength</td>
<td>Splitting strength</td>
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<td></td>
<td>Particle size</td>
<td>Laser granulometer</td>
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<td></td>
<td>Rheology</td>
<td>Rheometer, DIN EN 1015-3</td>
</tr>
<tr>
<td></td>
<td>Shrinkage</td>
<td>Strain gages</td>
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<tr>
<td></td>
<td>Compressive strength</td>
<td>DIN EN 1015-11</td>
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<td></td>
<td>Flexural strength</td>
<td>DIN EN 1015-10</td>
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<tr>
<td></td>
<td>Apparent density</td>
<td>DIN EN 1015-10</td>
</tr>
<tr>
<td></td>
<td>Adhesion strength</td>
<td>DIN EN 1015-12</td>
</tr>
<tr>
<td></td>
<td>Injectability</td>
<td>On cracked test specimens with subsequent analysis of the injection performance</td>
</tr>
<tr>
<td></td>
<td>Reactivity</td>
<td>Isothermal calorimetry</td>
</tr>
<tr>
<td></td>
<td>Frost resistance</td>
<td>Freeze-Thaw test</td>
</tr>
<tr>
<td></td>
<td>Salt crystallization test</td>
<td>Crystallization test with Na₂SO₄</td>
</tr>
</tbody>
</table>

Though from the manufacturer certain material properties are given, the detailed parameters important for the mechanical properties of the single materials will be determined in our labs. These include the tests and standards given in Table 5.5.

<table>
<thead>
<tr>
<th>Masonry composing material</th>
<th>Property</th>
<th>Standard, test procedure</th>
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</thead>
<tbody>
<tr>
<td>Earth mortar</td>
<td>Particle size</td>
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<td>Crystallization test with Na₂SO₄</td>
</tr>
</tbody>
</table>

Due to the extent of the experiments only one particular set of materials was considered for the study. A variation in materials properties (i.e. for simulating other material characteristics) will be the content of the subsequent numerical simulation of WP4.2.

Adobe blocks, earth mortar and rammed earth are ready-made products from the manufacturer. Cob, however, needs to be prepared in our labs. For the preparation of cob, we will use soil from the manufacturer of the other earthen products and mix it with straw fibres to a mass of plastic consistency. Mixing will be performed in a concrete mixer. After mixing 0.7 x 0.7 x 1.0 m³ high walls will be built up in the traditional way of cob building. After drying of the small walls the test specimens will be cut out by means of a saw.
5.2.3 Static experiments on small sized wall segments

The static experiments, as pointed out before, will be the basis for establishing the properties of the walls under compressive and shear load. In essence two types of tests will be performed in order to gain the various mechanical properties of the different wall types: Compressive and shear loading. The shear tests will analyze the de-bonding behaviour of the masonry wall segments (mortar/earth block) and the resistance of the monolithic wall segments towards shear forces. Additionally, cracked wall segments will be grouted and subjected to flexural and shear load test in order to analyze the mechanical performance of the injection mortar.

5.2.3.1 Wall segment specimens

For the static compressive experiments it is foreseen to produce wall segments of the size 0.50 x 0.52 x 0.11 m$^3$. The masonry segments are composed of 6 courses with a joint width of 20 to 25 mm and are in accordance with DIN EN 1052-1 (Fig. 5.3). The joint width was chosen bigger than recommended by the standard, taking into account the often large joint widths in historic adobe masonry. The monolithic walls will be built in the same size. Rammed earth walls will be produced with respective formworks. Cob wall segments will be cut from bulk specimen, which are built in typical cob style with raw earth, water and straw fibres (20 to 30 cm long). Figure 5.3 shows one of the bulk specimens after finishing and during the drying process. After drying segments in the size of 53 x 50 x 11.5 cm$^3$ will be cut from the bulk specimens, thus preserving the original texture of the cob.

After production the wall segments are stored in a climate room at 23 °C and 50 % RH for equilibrating the moisture content of the walls. The specimens will only be removed from the climate room shortly before the strength testing. Prior to the tests two double T-girders will be attached to the lower and upper side of the wall segments for introducing the compression forces into the specimens. Accurate parallelization of the girders will be done by a low strength cement mortar joint between the girders and the wall segment.

5.2.3.2 Compression and shear load testing

The tests will be performed using a universal testing machine according to DIN EN 1052-1. The compression tests will be performed with the levelled specimens where the smaller sides of the wall segments are parallel to the loading plates of the testing machine (with the masonry samples, parallel to the courses). From each wall type 3 to 6 specimen will be tested. The deformations will be monitored by strain gauges (LVDTs) parallel and in vertical load direction and visually by a novel approach consisting of a photogrammetric method. The latter visualizes the deformation of a specimen during the test in form of a movie with a maximum frame rate of 15 fr/s. From the readings of the strain gauges the modulus of elasticity is calculated.
The shear tests will be performed in compression mode following ASTM E 519. In order to induce shear forces the wall segments are turned 45 ° around the middle axis with one diagonal of the wall segment being perpendicular to the loading plates of the testing machine. For the experiments 3 to 6 specimens will be tested. The deformation of the wall segments will be monitored by strain gauges (LVDTs) and by the photogrammetric system. The latter will be performed exemplarily on one to two wall segments.

Shear tests will be performed on undamaged specimen and on pre-cracked specimens, which were grouted with lime based mortar developed in WP3.2. After the testing of the undamaged specimens the ones which did not completely fail will be reused for grouting with the injection mortar and tested again.

The testing program foresees the following test matrix:

1. Compressive testing of undamaged specimen – 3 to 6 samples (DIN EN 1052-1)
2. Shear testing of undamaged specimen – 3 to 6 samples (ASTM E 519)
3. Shear testing of artificially cracked and grouted samples (from 2.) – 3 to 6 samples (ASTM E 519)

The tests will therefore be performed with a minimum of 9 and a maximum of 18 samples for each wall type.

Additional tests will be performed concerning the bond behaviour between mortar and block units of adobe masonry. Those tests will include the determination of the initial shear strength and bond strength. Initial shear strength will be performed on specimens consisting of three block units which are bound together by the earthen mortar. The test will be carried out according to DIN EN 1052-3 (Fig. 5.4). Both data sets are required for the modelling approach in D4.2.

For analyzing the shear behaviour of the grouting mortar the same initial shear strength test will be performed as described before (Fig. 5.4). The tests will be performed 28 d after injecting the grouting mortar into the samples to give enough time for curing. Another specimen set will consists of wall segments of rammed earth and cob, which will be subjected to a three point bending test in order to produce a crack. The crack will be grouted in the same manner as before and cured for 28 days before shear testing.
5.2.4 Dynamic in-plane loading experiments with unidirectional horizontal sliding table

The dynamic loading experiments are aimed predominantly at investigating the in-plane shear behavior of earthen walls under a loading situation similar to that of real earthquakes. In contrast to cyclic in-plane dynamic loading experiments displacements are induced on the base of the walls (cyclic in-plane shear loading induces forces at the top of the wall). The experiments will primarily be focussed on unreinforced adobe masonry and monolithic earth walls as well as walls of the same type reinforced with anchors. The focus will be two main intervention methods: Increasing the earthquake resistance by intra- and extramural anchors. Besides the intervention methods the general response of wall elements to dynamic shear loading will be investigated. This is in particular interesting with wall elements made with the cob methods since there have not been reported any systematic results concerning the response of cob walls under dynamic loading.

5.2.4.1 Wall segment specimens

The specimens which will be tested consist of near real scale wall segments (1:1.4). The test specimens consist of rectangular wall segments of a size of 1.8 x 1.8 x 0.4 m³ without window or door openings. The wall segments will be built in adobe on a reinforced concrete platform and in the same manner as the small masonry segments previously described. On top of the walls a reinforced concrete beam will be placed and fixed to the underlying wall test specimen in order to provide a defined dead load.

5.2.4.2 Envisioned reinforcement techniques

Reinforcement techniques to improve earthquake response for modern buildings are fairly well known and already mirrored in numerous building codes. For historical earthen buildings, however, the knowledge and experience of suitable intervention techniques is still lacking. In the course of our activities we therefore envision the following intervention methods in order to retrofit historical buildings against earthquakes:

- Intramural anchors applied inside a bore hole and stressed by external plates or prestressed and then grouted.
- Extramural anchor pairs applied on the outside of the walls in joints of the adobe masonry or on the walls and stressed by external anchor plates
- FRP strips, which will be glued with an adhesive (either polymer or mineral binder) into the surface of the walls

The feasibility of all the techniques listed above needs to be investigated in form of preliminary tests. For intramural anchors the most suitable drilling method in earth will be determined. For the FRP strips suitable types of adhesives needs to be found not only concerning adhesion but also dissipation capability of energy.

5.2.4.3 Experimental program

The actual experiments will be performed at BAM on a multipurpose testing field. For the duration of the test a special setup in form of a sliding table will be installed with a programmable actuator for the horizontal displacement. The actuator can change its performance characteristics within the following limits: force between 63 to 400 kN, velocity between 0.2 to 2.0 m/s and displacement between 0.05 to 0.2 m. The harmonic periodical displacements in-plane will be dimensioned in frequency and acceleration close to the ones faced during earthquake events.

The rectangular wall segments will be used for three types of experiments:

- Testing the general response of the different types of walls towards dynamic horizontal loading.
- Testing the effect of reinforce adobe walls by intra and extramural anchors.
• Determine the influence of increased moisture content of the wall base. The actual experiments will be performed by applying specific loading patterns which simulate acceleration, velocities and frequencies similar to real earthquake events. During the experiments the displacement of the walls in relation to the shear forces will be monitored.

5.3 Partner n° 3 – ITAM: Bhutan case - Construction in Bhutan

5.3.1 Introduction and objectives

ITAM will perform a study on an actual architectural case from Bhutan on how to reinforce traditional architecture from this region against shear loads induced by earthquakes. Most of the monuments in the country date back to the 17th and 18th century but some of them date back as far as the 7th century. In addition to the architectural, aesthetical, historical, documentarily, and archaeological significance, most of the historical monuments in Bhutan have deep spiritual significance. One of the biggest problems related to heritage sites in Bhutan is the difficulty in the structural analysis of traditional buildings (Fig. 5.5):

• Almost all the traditional buildings are constructed using indigenous knowledge and materials
• There is no means of factual calculation to determine the stability of buildings due to their heterogeneous construction
• Almost all the massive structures have no foundation but are directly constructed on the underlying bedrock or ground
• Bhutan suffers from an acute shortage of qualified professionals in the field of structural analysis of monuments and historical constructions

As a consequence the conservation and structural retrofitting techniques in Bhutan are still mostly traditional and rudimentary. The idea of state-of-the-art conservation is at its developing stage and it is very urgent for Bhutan to become fully aware of modern ideas and techniques for conservation. Today Bhutan is carrying out renovation works extensively, which not necessarily comply to actual rules and charters for cultural heritage. Statistics show 70 renovations every year. At this rate there is danger of losing the authenticity of the architectural heritage of Bhutan quickly. The goal of the study is therefore to explore compatible and suitable techniques for retrofitting historical buildings in Bhutan against earthquake loads. The approach will combine modern materials with the least impact intervention methods.

5.3.2 Materials and material properties

The materials used will contain typical and traditional construction materials used for monuments in Bhutan and include the following:

• Rammed earth for masonry walls ("Bhutan case")
• Soil with a higher content of organic matter (humus)
• Typical historical irregular stone
• Earth mortar

The materials will be analyzed and their mechanical and physical properties determined.
5.3.3 Experimental Program

The experimental program is based on two types of dynamic load experiments: Tests on wall/masonry segments and tests on scaled down typical buildings.

5.3.3.1 Specimens

- Bhutan-typical masonry wall with and without retrofitting (reinforcement)-scale 1:1 (1:2)
- Model structure of a historical building from Bhutan (scale 1:10), see also Figure 5.5

5.3.3.2 Reinforcements

- Fibre reinforced polymer (FRP)
- Plastic nets
- Reinforcing elements (Fig. 5.6) include carbon fibres and in terms of strength parameters they rank in the first place among the types used: High modulus of elasticity in tension (up to 600 GPa) and high tensile strength (up to 4000 MPa) are close to the strength characteristics of the steel reinforcement used in concrete, and are therefore very suitable materials for reinforcement of these structures.
- Usually, the reinforcing elements in composite materials are applied to the external surface of the bonded reinforcement of existing structures. In some cases reinforcing elements are applied to the grooves in the structure.

5.3.3.3 Dynamic loading tests on shaking table

The test specimens will be exposed to basic dynamic identification tests on the shaking table. The test programme includes:

- Small scale tests under harmonic/random excitations.
- The different type retrofitted specimens will be tested under series of simulated earthquake ground motions.
5.4 Partner n° 4 and 16 – NTUA in collaboration with S&B: Stone Masonry Walls - Laboratory Experimental Campaign

5.4.1 Introduction and Objectives

The contribution of NTUA (Laboratories of Reinforced Concrete and Earthquake Engineering) in cooperation with S&B regards the behaviour of three-leaf stone masonry before and after grouting. This investigation includes testing of walltettes subjected to monotonic compression.
The type of masonry is selected to simulate one of the most common construction types used in historic structures in Europe and elsewhere.

The main objectives of this experimental campaign are the following:

(a) To check the efficiency of a new material (produced by S&B) in strengthening three-leaf masonry and to compare the performance of masonry with that exhibited by masonry strengthened using hydraulic lime based grouts. The performance of grouts will be checked in terms of compressive strength of masonry, as well as in terms of horizontal and vertical deformations. Actually, deformations are a key parameter, due to the fact that earthquakes impose to the structures deformations and not loads.

(b) To obtain data that will allow for constitutive models to be developed. Those models (adequately simplified) will be used in further WPs for modelling the behaviour of subassemblies and entire model structures.

5.4.2 Experimental Program

5.4.2.1 Materials

The first step of the experimental program includes the characterization of the materials that will be used both for the construction of the wallettes and for the two alternative grouts.

**Stones:** Limestone rubble stones (of compressive strength approximately equal to 100MPa) will be used for the construction of wallettes. Basic physical and mechanical properties of stones will be obtained experimentally.

**Mortar:** A low strength lime-pozzolan mortar will be used for the construction of wallettes. The characteristics of the mortar are selected taking into account the limited time available between the construction of wallettes and their testing. The properties of the mortar will be experimentally determined.

**Filling material:** Due to the fact that the properties of the material that fills the gap between the two external leaves of masonry govern the behaviour of masonry, 16 cylinders of filling material will be constructed (in parallel with the construction of wallettes). Their dimensions will be: diameter=250mm, height=500mm. Three of the cylinders will be tested as built, in order to determine the initial compressive strength of the filling material. Six cylinders will be injected with hydraulic lime based grout, whereas six cylinders will be grouted using the mix that contains the $\mu$-silica developed by S&B. The cylinders will then be tested to compression. The complete compressive stress vs. compressive strain curves will be registered, along with their horizontal dilatancy. The efficiency of grouts to fill the voids of the cylinders will also be examined after completion of tests.

**Grouts:** The two alternative mixes to be used in the experimental campaign will be characterized. In order to check the efficiency of the grout that contains the $\mu$-silica developed by S&B, a hydraulic lime based grout will also be used for the sake of comparison. For this purpose, a hydraulic lime based grout mix, already tested by NTUA in previous research work, will be used. All necessary laboratory testing (including sand column tests, compression tests, etc.) will be carried out. For the grout mix containing the S&B material, more detailed laboratory work will be needed. Actually, this material has proven its efficiency in mortars. As it has not yet been tested as component of grouts, it has to be carefully characterized. For this purpose, alternative trial mixes will be produced with various fine material percentages. The injectability, the fluidity and the stability of the mixes will be measured before selecting the mix proportions of the grout that will be compared with the hydraulic lime based one.

It should be noted that the entire procedure to be followed for the characterization of materials will be recorded, so that to serve as basic information for the drafting of Guidelines regarding the intervention technique of grouting.
5.4.2.2 Wallettes

Four wallettes made of three-leaf stone masonry will be constructed. Their dimensions are as following: Length=1.0m, Height=1.20m, Thickness=0.45m. Those dimensions are similar to those of previous specimens, thus allowing for direct comparison with available experimental data and (b) they allow for masonry wallettes being not dependent on scale effects. Within their thickness, the wallettes consist of two external leaves of stone masonry and one intermediate leaf of filling material, of practically equal thickness (~3x0.15m). The wallettes are constructed on a strong steel base (provided with stiffeners). An identical steel element is placed on top of each wallette. Thus, uniform distribution of compressive stresses is ensured, as well as practically zero deformations of the steel elements during loading of the wallettes.

5.4.2.3 Testing

The four wallettes will be tested in compression as built, up to their maximum resistance. Subsequently, they will be grouted (two wallettes per grout mix) and they will be re-tested to compression up to failure. During testing (before and after grouting) the vertical, the horizontal and the transverse deformations of wallettes will be recorded.

It should be noted that due to the major importance of the grouting procedure for the efficiency of that intervention technique, very detailed record of the entire work will be kept. Those data will serve the purpose of drafting Guidelines for the application of grouting technique.

Furthermore, the testing procedure (loading sequence and measurements) will be used (in parallel with other testing procedures) to draft the respective Guidelines for characterization of masonries and for checking the efficiency of grouting technique.

5.5 Partner n° 5 – POLIMI

5.5.1 Stone Masonry Walls - In-situ Experimental Campaign on wall quality (in cooperation with UNIPD)

5.5.1.1 Introduction and objectives

POLIMI has a long experience on the classification and study of masonry quality developed on site in different Italian regions (Tuscany, Sicily, Umbria, Lombardia, Liguria, etc.) and owns a database on stone masonry typologies and characteristics. Also following the requests of the Guidelines of the Italian Ministry for C.H. on the detection of masonry quality as a base for knowledge, has set up and proposed a methodology for onsite investigation of masonry quality base on non destructive (NDT) and Minor destructive (MDT) investigation techniques.

This methodology will be applied to some monuments in L’Aquila and other sites of Abruzzo in order to study the masonry characteristics and the possibility of applying to it different repair techniques as grout injections, re-pointing, etc. The aim is to control the quality of the peculiar masonry typologies in the Region as it was already done just before the earthquake in Sulmona near L’Aquila.

5.5.1.2 Experimental program on the onsite detection of masonry quality

The proposed experimental methodology will be performed locally according to the following steps:

a) choice of the strategic points of the wall where to carry out the investigation by simple visual inspection if the masonry is not rendered or by the use of NDT (thermography, radar)

b) survey of the masonry texture in prospect by detaching if necessary 800x800mm of render

c) sonic pulse velocity test by transparency on a grid of 7x7 points minimum and representation of a map of velocity distribution (the highest peaks of velocity can indicate the presence of transversal connections in the section
d) single flat jack test to define the local stress component in compression
e) double flat jack test represented in the stress-strain diagram where also the result of the single flat jack is reported
f) small dismantling by extraction of one or two stones up to the 2/3 rd inside the wall section and drawing of the section prospects by defining the type of section (one, two or more leaves with or without filling, sampling of mortar, brick or stones
g) reconstruction of the locally dismantled wall
h) characterisation of the materials by laboratory chemical, physical and mechanical tests if possible.

By small dismantling also the wall-to-wall, floor to wall connection will be detected in several C.H. buildings

The experimental program will be developed on C.H. buildings (if allowed and possible) as the Spanish Fortress, the Churches of S. Biagio in Amiterno and S. Giuseppe dei Minimi in L’Aquila and the church of St. Giusta in Bazzano.

Where the on-site mechanical tests will not be possible the masonry quality will be indirectly detected by NDT (sonic, thermography and radar tests)

5.5.2 Stone Masonry Walls - In-situ Experimental Campaign on wall repair by injection (in collaboration with UNIPD)

5.5.2.1 Introduction and Objectives

The contribution by POLIMI is given on the masonry panels chosen by the Department of Structural and Transportation Engineering of the University of Padua which include a series of in-situ diagonal tests and NDT applied to two leaves stone masonry walls typical from the region of Abruzzo (Italy). These walls (test specimens) were chosen among the damaged buildings on the localities of Onna, S. Eusanio Forconese and Tempera (Abruzzo, Italy). After characterisation the walls were injected with various types of commercial grouts and tested again after injection POLIMI was collaborating to the on site survey of the walls and of their transversal sections and to the detection of the grout penetration. Furthermore POLIMI will carry out tests on injectability and characterisation of the grouts.

Furthermore other walls will be in the future chosen and injected and their injectability studied by POLIMI on site by NDTs so as the characterisation in laboratory of grouts proposed by new producers.

5.5.2.2 Experimental Program

Basically, the further investigation by POLIMI to the experimental in-situ campaign will consist of:

1. Finding safe and suitable walls to perform the experimental tests;
2. Perform preliminary sonic pulse velocity characterization of the unreinforced walls
3. Injection of the walls with an hydraulic lime mixture up to half its length, measuring the quantity of grout in order to attain an estimation of the void percentage on the chosen walls;
4. Post-injection sonic characterization of specimens
5. Laboratory tests to characterize before the injection the proposed grouts and test their injectability into the chosen masonries applying the methodology proposed by POLIMI in the past (use of cylinders built in laboratory and injected).
5.5.3 Experimental program on the durability of repairs by composite materials

5.5.3.1 Introduction and objectives
The activity is focused on investigating the durability of masonry repaired by composite materials. Even if extensive research has been carried out on the mechanical properties of masonry repaired by FRP, practically no research has been carried out on the durability to aggressive environments with high humidity, temperature variation, presence of salts.

The aim of the research is to check the possible debonding of the FRPs from masonry, due to previous mentioned causes.

5.5.3.2 Experimental program
Specimens
The specimens are single solid bricks or small assemblages simulating diffused masonry textures as: a) soldier course specimens, with courses of bricks laid with the long sides upright (as in thin vaults), b) running bond specimens simulating common textures of masonry walls.

Samples for the crystallisation tests will be built by the research partners and sent to POLIMI for testing. Available materials at UniPD are solid bricks by Sant’Anselmo called Mattoni Pieni Giallo Macchiato, Sant’Anselmo A001GM MACCHIATO; compressive strength f_c=16 N/mm^2) and natural hydraulic lime mortar Tassullo T30V (M5). The chosen repair materials at UniPD are Carbon Fibers provided by BASF Construction Chemicals Italia spa divisione Nord-Est Italia.

Single brick specimen
Figure 5.7 shows the solid brick sample (standard dimensions 25.0×12.0×5.5 cm) for thermal ageing and adhesion-absorption controls by pull-off tests.

To the specimens prepared for thermal ageing controls and for adhesion-absorption controls, FRP strips 80 mm wide and 210 mm long will be applied. The 80 mm width is required by the test procedures and equipment according to ASTM C 1583 – 04 standard, which considers a diameter of 50 mm for the glued device. On the sample in Figure 5.7, two pull out tests can be carried out.

The pull off tests as control of the effectiveness of repair will be carried out also on the other types of specimens (soldier course and running bond) before and after the durability tests.

Soldier course specimens
Figure 5.8 shows the geometry of soldier course specimens for crystallisation tests with an overall dimension 25×25×5.5 cm³. This texture simulates, for examples, thin masonry vaults. FRP strips 50 mm wide and 210 mm long will be applied in four different geometrical configurations. In a fifth type, FRP textile covers the whole free surface of the specimen. The five configurations simulate the possible application on real case-histories. The 50 mm width of the CFRP strips was selected according to common application and for consistency to other tests.

Fig. 5.7 - Specimens for the thermal ageing and adhesion-absorption control by pull-off tests on single bricks (left), and possible localisation of the tests (right).
Fig. 5.8 - Soldier course specimens for crystallisation tests. Plain sample (only brickwork) (a), with FRP strips geometry in configuration 1 (b), configuration 2 (c), configuration 3 (d), configuration 4 and CFRP textile covers the whole free surface of the specimen (e).

Running bond specimens

Figure 5.9 shows the geometry of the running bond specimens for crystallisation tests with an overall dimension 25×20.5×12 cm³. This texture simulates the common texture of masonry elements.

The mortar joint shift allows the optimisation of the test campaign, investigating two reinforcement geometries in a single specimen, Fig. b) and d).

FRP strips 50 mm wide and 185 or 210 mm long will be applied in four different geometrical configurations. In a fifth type, CFRP textile covers the whole free surface of the specimen (Fig.). The five configurations simulate the possible application on real case-histories.

The 50 mm width of the CFRP strips was selected according the characteristics of common materials and of the other tests.
Fig. 5.9 - Running bond specimens for crystallisation tests. Plain sample (only brickwork) (a), with FRP strips geometry with configuration 1 (b), configuration 2 (c), configuration 3 (d), configuration 4 and FRP textile covers the whole free surface of the specimen (e).

**Test description**

**Crystallisation test**

The test simulates the effect of the salt crystallisation on the bonding. The horizontal maximum dimensions of the specimens should fit the shape in Figure 5.7, about 25x25 cm². The number of sample should be at least 2 for each strengthening configuration.

The test procedure and specimen details are reported in the Document of the RILEM TC 127-MS.

**Adhesion control after thermal ageing**

FRP reinforced specimens shall be subjected to thermal cycles. The specimens should have a moisture lower of 12% and cured in control environment (20°C and 50% RH) for 24 hours.

Artificial ageing shall be carried out as follow (at 60% RH):

a. +20 to +70°C in 1 h (0.8°C/min)
b. 3 hours at 70°C
c. +70°C to -10°C in 1:30 h (1.0°C/min)
d. 3 hours at -10°C
e. -10°C to +70°C in 1:30 h (1.0°C/min)

Repeat from (b.) with an automatic control for at least 40 cycles.

The number of specimens is 2 for each strengthening configuration (see par. 4 and Table 2).

**Adhesion-absorption tests**

To check the influence of humidity on bond, FRP is applied to masonry after water saturation of the sample

Three conditions should be tested:

i) in laboratory environment condition (20°C and 50% RH) (about 5-6% of water content for the chosen bricks),

ii) in saturated conditions (about 22% of water content for the chosen bricks),
iii) in intermediate degree of water absorption between saturation and environment condition (about 14% of water content for the chosen bricks). This last condition can be estimate by the mass changes, weighting the dry sample after water saturation and at different step during the water evaporation. If the drying is in stove, before the FRP application wait for about 20 minutes before applying the fibres in order to re-equilibrate the sample with the laboratory conditions.

Then pull-off tests shall be performed at laboratory environmental conditions.

The global number of specimens is 6, 2 for each test condition and each strengthening configuration.

5.6 Partner n° 6 – UMINHO

5.6.1 Experimental program on timber frame walls

5.6.1.1 Introduction and objectives

Given the complexity of the behaviour of timber structures, especially as concerned traditional timber joints, and due to the lacking of expertise on the mechanical behaviour of Pombalino timber frame walls (Lisbon downtown buildings), sound research on the monotonic and cyclic behaviour is needed. The increase on knowledge on mechanical performance of timber brace walls is fundamental to provide guidelines for its rehabilitation and for the accurate assessment of the vulnerability of ancient Pombaline masonry buildings. Timber frame walls can even be seen as a strengthening technique of ancient masonry structures located in high seismic hazard regions.

The main goals of this test campaign are to characterize the in-plane behaviour of timber frame walls based on in-plane static cyclic tests and to test traditional timber connections and definition of appropriate strengthening techniques for the timber joints that enhance the cyclic behaviour the timber frame walls. Distinct variables influencing the lateral behaviour of the timber walls and the connections are to be analyzed.

5.6.1.2 Materials and material properties

The following materials will be used in the tests:

- Timber: Chestnut, oak and pitch-pine will be used to construct the timber frame walls.
- Mortar: A low strength lime mortar will be used for the plasters of the of timber frame walls. The properties of the mortar will be experimentally determined.
- Filling material: Two different infill materials will be used, namely rough brick masonry and a composite gypsum and cork based material. These two types are the common used infill materials of the Pombaline timber frame walls.

5.6.1.3 Experimental program

The experimental program has been divided into three main tasks:

Task 1: Testing of timber frame walls

The first task concerns the experimental characterization of timber frame walls. This task aims at obtaining the resisting and dissipative properties of the Pombaline timber frame walls, which are considered to be the main dissipative elements in ancient Pombaline masonry structures. It is intended to carry out cyclic tests on two different typologies of timber brace walls, with distinct three vertical pre-compression levels and with two distinct infill materials, namely rough brick masonry and a composite gypsum and cork based material. The latter possibility of infill material aims at proposing timber frame walls as alternative partition walls. The planning of experimental campaign also foresees the update of the test setup existing at University of Minho and the definition of the loading protocol. The analysis of results includes the comparative study of the distinct walls in
terms of failure modes, force-displacement diagrams and lateral resistance. Additionally, seismic performance should be evaluated based on bilinear elasto-plastic idealization of the experimental envelope from which information on the ductility and equivalent elastic stiffness, energy dissipation and viscous damping.

**Task 2: Testing of timber joints**

This task aims at characterizing and at giving strengthening strategies for improving the timber connections of the timber frame walls in order to enhance their resistance to seismic action. The experimental program aims at characterizing the typical timber joints existing in timber frame walls in order to define or update a hysteretic model and give experimental parameters to calibrate the hysteretic model.

**Task 3: Testing Strengthened timber frame walls and joints**

The strengthening strategies should consider innovative FRP composites and traditional strengthening with nails or iron steel bars can be also envisaged. With this respect, the lateral resistance and hysteretic behaviour, namely the dissipation of energy, stiffness and strength degradation are the main parameters to be analysed. This task aims at derive an hysteretic model for the timber frame walls and for the joints.

- **Specimens**
  - 3 Timber frame walls without infill material.
  - 3 Timber frame walls with rough brick masonry infill material.
  - 3 Timber frame walls with a composite gypsum and cork based infill material.
  - 3 Timber frame walls without infill material and with FRP strengthening.
  - 3 Timber frame walls without infill material and with traditional strengthening.
  - 3 Timber frame walls with infill material and with FRP strengthening.
  - 3 Timber frame walls with infill material and with traditional strengthening.
  - 9 Timber joints with different traditional connections.
  - 9 Timber joints with different traditional connections and with FRP strengthening.
  - 9 Timber joints with different traditional connections.

- **Reinforcements**
  - FRP composite materials.
  - Traditional timber strengthening techniques, such as nails or iron steel bars.

- **Experiments**
  - In plane static cyclic combined flexural and compression behaviour for the frame walls.
  - Static cyclic compression/tensile tests on timber joints.

### 5.6.2 Experimental program on grout injection of rammed earth

#### 5.6.2.1 Introduction and objectives

Earth constructions constitute an important and significant part of the built heritage in the world, and Portugal is no exception, despite that building with earth is not usual anymore. Although having in mind this fact, it is quite surprising the quantity of existing earthen buildings in the region of Aveiro, where the main earthen building typology is adobe (Silveira et al., 2010). Rammed earth is also an earthen building typology present in the national territory, mainly, in the regions of Alentejo
and Algarve. Due to the cultural and historical importance of these buildings, efforts are being currently directed to research on their preservation and rehabilitation.

Grout injection is currently seen as a repair/strengthening solution with enhanced potential for earth constructions. However, the non-reversible character of grout injection and the principles that regulate the intervention on historical buildings demanded research on grouts compatible with earth constructions to be carried out. As result, grouts that have earth as one of the main components (mud grouts) were adopted and studied by some researchers and conservation practitioners (e.g. Roselund, 1990, Vargas et al., 2008). Nevertheless, the existing knowledge about this type of grouts is still insufficient. The heterogeneity of earth as a material and the tight requirements of a grout demand a multi-disciplinary approach to be carried out. Thereby, in the current experimental program it is aimed at evaluating the efficiency of mud grouts applied to unstabilized rammed earth constructions. The multi-disciplinary character of this study will require the fully characterization of the raw materials to be used. Moreover, special focus will be given to the characterization of the properties of the mud grouts. The efficiency of the mud grouts to be studied will be evaluated through mechanical tests to be carried out on wallets and walls representative of rammed earth constructions from Portugal.

5.6.2.2 Materials and materials properties

The rammed earth specimens of the main experimental program will be built by using a soil representative from Alentejo. In first place, the suitability of the soil for rammed earth construction will be evaluated according to existing worldwide regulations and recommendations. This will require the characterization of the particle size distribution (PSD) and plasticity limits of the current soil. Furthermore, the influence of the PSD on the mechanical properties of the rammed earth will be evaluate by changing the PSD of the soil through addition of sand or/and gravel in different proportions. Rammed earth blocks will be prepared and tested under compression in order to evaluate their mechanical properties. Moreover, visual inspection of the blocks will provide indication on the shrinkage behaviour of the rammed earth walls and wallets. The quantity of blocks to be tested will depend on the PSD of the soil. The final PSD of the soil to prepare the rammed earth walls and wallets will be defined from these tests.

Two types of mud grouts are intended to be tested: an artificial mud grout and a natural mud grout. The solid fraction of the artificial mud grout will be composed by kaolin and limestone powder, and the solid fraction of the natural soil will be composed by the soil to be used on the construction of the rammed earth specimens, which will be previously sieved and the PSD will be corrected by addition of limestone powder. Sodium hexametaphosphate (HMP) will be used as deflocculant/dispersant in both grouts. In order to define the composition of the two mud grouts (water content, PSD and quantity of added HMP) a composition study will be carried out. Table 5.6 presents the tests intended to be carried out in this phase. There will be evaluated properties of the mud grouts both in the fresh (mostly rheological properties) and hardened state (mostly mechanical properties) by resourcing to simple tests. Further characterization of the selected compositions will be carried out in the main experimental program.

Tab. 5.6 – Test concerned with the definition of the composition of the mud grouts.

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimens</th>
<th>Evaluated property(ies)</th>
<th>Standard(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSD</td>
<td>Kaolin and limestone powder</td>
<td>Particle size distribution</td>
<td>ASTM D 422</td>
</tr>
<tr>
<td>Marsh cone</td>
<td>Mud grouts (several compositions)</td>
<td>Flow time</td>
<td>ASTM C 939</td>
</tr>
<tr>
<td>3-point bending</td>
<td>Hardened mud grouts</td>
<td>Flexural strength</td>
<td>EN 1015-11 (1999)</td>
</tr>
<tr>
<td>3-point bending</td>
<td>Shallow rammed earth blocks</td>
<td>Adhesion</td>
<td>-</td>
</tr>
</tbody>
</table>
5.6.2.3 Main experimental program

In this phase of the experimental program several walls and wallets will be tested. These specimens will be tested in compression, diagonal compression and cyclic shear-compression as it is shown in Table 5.7. For each kind of test it is expected to test at least 6 specimens. Subsequently to the testing, these specimens will be repaired by injection using the selected mud grouts. The rammed earth specimens will be retested after the complete drying.

Tab. 5.7 – Test to be carried out on the rammed earth specimens.

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimens</th>
<th>Evaluated property(ies)</th>
<th>Standard(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression</td>
<td>Wallets</td>
<td>Compressive strength</td>
<td>EN 1015-11 (1999)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E. modulus</td>
<td></td>
</tr>
<tr>
<td>Diagonal compression</td>
<td>Wallets</td>
<td>Indirect tensile strength</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shear strength</td>
<td></td>
</tr>
<tr>
<td>Cyclic shear-compression</td>
<td>Walls</td>
<td>Behavior under shear</td>
<td></td>
</tr>
</tbody>
</table>

As mentioned previously, further characterization of the mud grouts will be also carried out. Eventually this detailed characterization will dictate small adjustments of the composition of the selected mud grouts. Thereby it will be carried out before repairing the rammed earth specimens. These tests are presented in Table 5.8 and aim at evaluating the main grout properties, such as: rheology, stability, strength and bond.

Tab. 5.8 – Characterization tests on the selected mud grouts.

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimens</th>
<th>Evaluated property(ies)</th>
<th>Standard(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rheometry</td>
<td>Fresh mud grouts</td>
<td>Flow curve</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flow time-dependent behaviour</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>mud grout cylinders (Ø50x100mm³)</td>
<td>Compressive strength</td>
<td>NBR 13279 (1995)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E. modulus</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C. Poisson</td>
<td></td>
</tr>
<tr>
<td>Direct tension</td>
<td>mud grout cubes (50x50x50mm³)</td>
<td>Tensile strength</td>
<td></td>
</tr>
<tr>
<td>Sand column injection</td>
<td>Fresh mud grouts</td>
<td>Penetrability</td>
<td>AFNOR P 18-891 (1992)</td>
</tr>
<tr>
<td>Bleeding</td>
<td>Fresh mud grouts</td>
<td>Bleeding (stability)</td>
<td>ASTM C 940 (1989)</td>
</tr>
<tr>
<td>Density variation</td>
<td>Fresh mud grouts</td>
<td>Sedimentation rate</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(stability)</td>
<td></td>
</tr>
<tr>
<td>Direct tension</td>
<td>Sandwich specimens</td>
<td>Tensile strength</td>
<td></td>
</tr>
<tr>
<td>Shear box</td>
<td>Sandwich specimens</td>
<td>Shear strength</td>
<td></td>
</tr>
</tbody>
</table>
5.6.3 Experimental program on bond durability of composite materials applications to masonry structures

5.6.3.1 Introduction and objectives
Modern composite materials as fibre reinforced polymers (FRP), steel reinforced polymers (SRP) and natural fibres have been increasingly accepted as effective repairing or strengthening technique of masonry structures due to high mechanical properties and light weight. The efficacy and reliability of this technique intrinsically depends on bond behaviour between the composite materials and the masonry substrate. In this regard, investigation into bond durability is a key issue since service failures due to inaccurate characterization of aging bond response may result in costly repair or premature components replacement.

Analysis of long-term durability of FRP applications involve laboratory and field testing, accelerated tests and advanced computational models, but research in this area is still few (Karbhari et al. 2000). Only limited studies have been conducted on durability of FRP externally applied to masonry (Briccoli Bati and Rotunno 2001, Aiello and Sciolti 2004, Desiderio and Feo 2005). Available laboratory-based research on FRP composites needs therefore to be supported by materials characterization and experimental activity on FRP- structural components (Karbhari et al. 2003, ASTM E632 1998) in view of: (i) reliable durability design of FRP-strengthened components, (ii) prediction of long-term response thorough accelerated ageing tests and (iii) developing feasible computational models capable to capture relevant degradation mechanisms and damage.

Moreover, development of standardised accelerated ageing procedures to establish long-term performance of FRP-masonry components is still strongly required as already recognized by the Technical Committee 223-MSC “Masonry strengthening with composite materials” from RILEM Committee. In addition, while methods to evaluate fundamental responses of composite materials are fairly well established for specific degradation mechanisms acting alone, the potential for synergistic effects among mechanisms is not completely understood.

Among the major tasks for characterizing the durability and long-term behaviour of FRP-masonry systems is the development of reliable numerical models that account for the effects of stresses and environmental exposures. The ability to predict the evolution of the time-dependent behaviour of the interface as a function of environmental changes over the lifetime of a structural component is still critical at design stage. Moreover, few studies are available for the prediction of long-term durability of composites based on the coupled effects of all these non linear mechanisms (Martin et al. 2009, CIB W080). Actually there is the need to properly account for nonlinear mechanisms such as moisture, aging effects and the mechanical degradation in FRP composites.

The main objective of the research is to build a systematic knowledge based for long-term durability behaviour of bond between masonry and fibrous composite materials thorough relevant experimental activity. Accelerated exposures and testing will be accomplished through a number of schemes, depending on several aging mechanisms and environmental variables. Differences in bond behaviour will discussed with reference to freeze-thaw and hygrothermal environmental conditions. The quantitative analysis of bond behaviour between masonry and composite materials will be performed thorough shear bond tests and pull off tests, and comparisons will be carried out between exposed and un-exposed specimens. The gathered data will include interface long-term constitutive laws, physical and mechanical data of materials and masonry specimens.

5.6.3.2 Materials and material properties
Brick masonry specimens will be built with materials characterized by low mechanical properties, in order to represent materials of historical constructions. The masonry texture shall replicate typical masonry arrangements. The composite materials will include: glass fibers and steel fibers. Epoxy-based resins will be used for both glass (GFRP) and steel fibers (SRP). Premixed or hand-made mortars will be used also as matrix for steel fibers (SRG) The strips will be applied according to dif-
different geometrical configurations, in order to simulate the possible application on real case-histories. Fundamental ageing characterization of basic materials, namely bricks, mortars, resins, FRP coupons and masonry specimens will be achieved.

5.6.3.3 Experimental program
The experimental programme has been divided in four main tasks:
1. Freeze-thaw accelerated tests
2. Hygrothermal exposure tests
3. Post-ageing shear bond tests
4. Characterization of the basic materials

Task 1. Freeze-thaw accelerated tests
Task 1 focuses on the effects of freeze-thaw conditions on the composite material-masonry interface. The specimens include brick samples with a composite fabric 50 mm wide applied on a single side, to be tested under shear bond (see Fig. 5.10), and brick wallets for pull-off tests. The total number of specimens is summarised in Table 5.9.

![Fig. 5.10 - Composite - brick samples](image)

![Tab. 5.9 - Shear bond tests](image)

<table>
<thead>
<tr>
<th>Type of specimen</th>
<th>Composite material</th>
<th>Number of specimens per each type of composite material</th>
<th>N. tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single brick</td>
<td>GFRP, SRG, SRP</td>
<td>30</td>
<td>15</td>
</tr>
<tr>
<td>Wallet</td>
<td>GFRP, SRG, SRP</td>
<td>30</td>
<td>15</td>
</tr>
</tbody>
</table>

Accelerated freeze-thaw tests will be performed by varying the following parameters: the number of cycles per day, the exposure time, heating and cooling rates. The specimens will be cured in the following laboratory environment condition: T=20±5°C and RH=60±5% for at least 24 hours. The water content for the chosen bricks before testing will be lower of 12%.

Thermal cycles will be carried out as follows:
1st day: 1h (+20 to +70); 3 h; 1,5h(+70 to -10); 3h; 1,5h(-10 to +70); 3h; 1h (+70 to+20).
2nd day: 0.5h(+20 to -10);3h; 1,5h(-10 to+70); 3h;1,5h(+70 to -10); 3h;0.5h(-10 to +20).

A sketch of the exposure path is illustrated in Fig. 5.11. The specimens will be subjected to unidirectional exposure for at least 30 days.

![Thermal cycles over the first day](image1)

![Thermal cycles over the second day](image2)

**Fig. 5.11 - Thermal cycles over the first day (a), and the second day (b).**

**Task 2. Hygrothermal exposure tests**

The task is aimed at investigating both short and long-term effects hygrothermal exposure on FRP-masonry interface behaviour.

Most of FRP materials used in civil applications can come in contact with moisture and various solutions, either due to natural causes or location. The primary effect of the absorption that will be investigated is on the resin itself which is expected to cause reversible and irreversible changes in the polymer structures. In particular, changes in the glass transition temperature of the resin will be monitored. Moreover, degradation at fibre levels will be investigated for carbon, glass and natural fibers. In this case, the monitored properties will be fibre-matrix debonding and matrix microcracking in the composite.

Extensive ageing tests will be also carried out on strengthened bricks and masonry specimens accounting for the synergistic action of moisture and temperature variations. In particular, moisture exposure will be accompanied by temperature cycling and, in this case, particular attention will be paid to the following conditions: (i) the effect of moisture absorption of the bricks during an initial phase of immersion in water; (ii) the effect of low temperature during the freeze part of the cycle.

At the end of the exposure periods, inspection will be carried out in order to check failure of the specimens, as well as the matrix microcracking and fibre-matrix debonding.

**Task 3. Post ageing shear bond tests**

This task deals with post ageing tests on FRP-masonry systems and with the experimental characterization of the masonry and its basic materials, the fibres and the resin.
With the aim to investigate the performance of FRP-masonry interface under accelerated environmental conditions, static shear bond tests and pull-off tests will be performed as follows:

- Single-shear bond tests and pull-off tests will be carried out on strengthened bricks and masonry prisms/wallets after aging exposure. These post-ageing tests will be carried out to assess the bond behaviour in different test configurations in terms of durability and long-term performance.

- Single-shear bond tests and pull-off tests will be carried out on strengthened bricks and masonry prisms/wallets not subjected to exposure conditions. The specimens will be tested at the same age of those subjected to environmental exposure.

In shear bond tests, video camera monitoring systems will be used for complete interrogation of the interface deformations and debonding phenomenon along the entire interface length, and therefore providing valuable information about the mechanics of interface. The values of interface law parameters (e.g. maximum transmissible shear, energy fracture) will be used for advanced FE analysis of the bond-slip behaviour of the specimens.

Task 4. Characterization of basic materials

Durability depends intrinsically on the constituents used in the FRP-masonry systems, namely neat resin, FRP coupons, mortar, bricks and unstrengthened masonry prisms. Therefore, with the aim to develop a reliable database of material properties, a set of freeze-thaw tests will be carried out, in order to monitor temperature and moisture profiles within the materials itself, along with progression of damage. Both mechanical and physical properties will be investigated. A batch of specimens will be tested after each exposure condition envisaged in Tasks T1 and T2. Another batch will be tested under laboratory environmental conditions, at the same age of those subjected to exposure. From comparisons, changes in physical and mechanical properties will be detected.

For the FRP constituents, changes in thermo-mechanical properties will be monitored. Analysis also includes response changes in the matrix due to temperatures above its cure temperature. This is a key point since working environments over a certain temperature range can cause an increase in viscoelastic response of the resins, a consequent reduction in elastic mechanical performance levels and, in a number of cases, an increased susceptibility to moisture absorption.

5.6.4 Experimental program on direct shear tests

Aiming at characterizing in detail the bond behaviour of masonry strengthened with composites materials as well as the most common collapse modes, an experimental program has been prepared in collaboration with Rilem Technical Committee 223-MSC, coordinated by the partner UNIPD. Four different types of reinforcements (carbon, glass and basalt fibres and steel cords, bonded with an appropriate epoxy) will be applied to a clay brick support. The setup will based on a single-lap shear testing device. In total 24 specimens will be tested.

5.7 Partner n° 10 – ENA

The contribution of ENA to WP4.1 will consist in mechanical and physical tests on traditional clay bricks which are utilized in traditional constructions in the Medina of Fez. The goal is the characterization of typical bricks from this area concerning their mechanical material properties in the context of further numerical modelling.

The main test consists in the uniaxial compression tests. These tests aim at obtaining the resistance of the block (Rc), Young’s modulus (E) and the percentage of deformation limit ($\varepsilon_{lim}$).
5.8 Partner n° 11 – CDCU: Case study Meqaad Radwan, Ottoman Period (1650 AD.)

5.8.1 Introduction and objectives
The contribution of the Conservation Department of Cairo University includes an applied study which is performed on an actual architectural case from Cairo, Egypt. The building named Meqaad Radwan dates back to the Ottoman Period (1650 AD.). This monument was dramatically affected by the 1992 earthquake. Figures (5.12) show parts of the induced failure on the case study. The main objectives of CDCU’s contribution is to identify the original building materials of the Meqaad Radwan building, to characterise their physical and mechanical properties and to define the deterioration aspects prior to reinforcement measures, reassembly and conservation of walls decorated with valuable mosaics.

5.8.2 Materials and material properties
The materials used for testing contain original materials used for the Meqaad Radwan case study and including the following:
- Limestone
- Fired bricks
- Lime mortar.
- Gypsum mortar.
- Ash mortar “QUSRMIL”
Ash mortar "Qusrmil" contains ash which resulted burning the raw materials in kilns, usually fuelled by waste matters, etc. The binder was added as a cementing material of the mortar. It is usually mixed with sand and sometimes lime. The sulphate content should not exceed 0.5 %. This mortar was very common in historical Islamic architecture in Egypt, specially the brick-work masonry.

5.8.3 Experimental program
The experimental programme is divided into two main sections.

5.8.3.1 Testing
It is composed essentially of compressive strength tests, porosity and density determination of stones and fired bricks specimens which allow characterizing the physical and mechanical properties of the original building materials.

5.8.3.2 Analysis
It aims to identify the chemical composition of stones, mortars and plasters using X-ray diffraction analysis (XRD). XRD analysis will be performed on the following materials:
- Stones (test specimens were chosen from the damaged limestone of the real case study).
- Fired brick (test specimens were chosen from the damaged fired brick of the real case study).

5.8.3.3 Further experimental work
- Reinforcements
- Injection of the walls with a hydraulic lime mortar.
- Reassembly of fallen mosaics units and fragments using lime mortar after cleaning and extraction of efflorescent salts.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

5.9 Partner n° 18 – MONUMENTA

5.9.1 Materials and materials properties
- Stone walls (3 leaves)
- Timber framed walls with cyclopean stone masonry filling (used in "pombalino" style buildings)
- Rammed earth

5.9.2 Experimental program
- Specimens
  - Foundation walls – column between empty spaces (such as windows/others)
- Reinforcements

The exterior facade of Meqaad Radwan. Horizontal wooden tie beam above column capitals connecting outside stone wall

Vertical cracking of the walls in the second floor (Fired or red brick and coating layer (lime gypsum plaster)

Cross section of a stone wall

Wood in the construction of wall and ceiling of the first and second floor

Wooden ties inserted within a brick wall

Fig. 5.12 – The Meqaad Radwan Building.
- Glass fibre and epoxy resin composite, with confinement connectors (steel anchors or CIN-TEC anchors).
- Polypropylene net, covering both wall surfaces, along with confinement connectors (transversal) or superficial fixing devices (RICHTERGARD system).

**Experiments**

- In-plane and out-of-plane loads in reinforced walls, reproducing earthquake induced damages, in order to obtain design guidelines for the strengthening techniques listed.
- Pull-off tests on rammed earth walls (low-intrusive) to obtain correlation with compression laboratory tests.
6 REFERENCES


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