Deliverable 5.1

Specification for laboratory specimens and testing strategies on floors and vaults

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1 INTRODUCTION

1.1 DESCRIPTION AND OBJECTIVES OF THE WORKPACKAGE

One of the main aims of NIKER project is to develop and validate intervention technologies that are based on the collaboration of traditional and innovative materials, to be applied on various types of structural elements. In particular, WP5 focuses on those elements that constitute horizontal diaphragms of historic constructions, such as traditional floors, roofs and vaulted systems. During WP5, the envisaged intervention techniques will be tested and numerically modelled with the final aim of establishing design procedures that optimize their use on existing buildings.

Hence, the goals of WP5 are:

- defining adequate and feasible intervention technologies for horizontal structural elements, also related to catalogues and requirements described in WP3;
- defining and improving laboratory procedures for evaluating the intervention technologies and give specifications for laboratory specimens;
- carrying out the necessary tests to characterise the experimental behaviour of original and strengthened wooden floors and roofs and masonry vaults, in order to obtain information on the system performance and the main constitutive laws relevant for modelling;
- numerically simulating the experimental behaviour and perform parametric assessment to seek for structural limitations or define optimized design procedures.

1.2 OBJECTIVES AND STRUCTURE OF THE DELIVERABLE

The main goal of this report is the presentation of a short overview on the role of horizontal diaphragms in the overall behaviour of historic buildings. This includes a recall to main structural types, their mechanical behaviour, the available strengthening techniques and the existing experimental techniques, used to derive information of the mechanical parameters and structural performance of horizontal diaphragms in their original state and after strengthening. Finally, a short overview of available models and design provisions for horizontal diaphragms, when available, is given. This intends to set forth and be an introductory work for planning the experimental program foreseen in WP5, dealing with the analysis and validation of the different solutions for strengthening horizontal structural elements. Hence, in the final chapters, this deliverable describes the test programs to characterize the constitutive behaviour, before and after strengthening, of horizontal elements, planned by the partners at this stage.
2 WOODEN FLOORS

2.1 INTRODUCTION

In terms of typology, on a wooden floor it’s possible to distinguish two main parts, with different structure and function, the supporting frame and the planking, (Angotti et al., 2005).

Structurally a floor has to:

- support the vertical loads without collapsing and without excessive deformation, (the criteria of out-of-plane strength and stiffness);
- have adequate in-plane stiffness in order to distribute the loads to the vertical structures;
- ensure an effective connection to the perimetric vertical structures in order to ensure a box type behaviour of the entire building.

2.1.1 Wooden floor typologies

In what concerns residential buildings under service loads the wooden floors can be divided into two main categories:

i. with the main bearing frame equal to the length of the span to be covered;
ii. with the main bearing frame with a length inferior to the span that needs to be covered;
iii. new generation wooden floors.

To the first group belong the simple and bidirectional frame floors and the composed frame floors, Figure 2.1. While the second group includes: Serlio type floors, (Figure 2.2), compartmented floors, polygonal floors, ray floors and all the floors that have a bearing frame composed by non-parallel beams, crossed in different directions and between them. Apart from these types there are also the new generation wooden floors, which are executed using laminated wood and wood-based composite materials such as Parallam, Intrallam or Microllam, Figure 2.3.

Figure 2.1 - Floors with the main bearing frame equal to the length of the span to be covered. (a) Simple unidirectional floors, (Munafò, 1990). (b) Bidirectional floors with double frame, (Munafò, 1990). (c) Mixed floor with timber beams and tile vaults, (Ceccotti, 2007).
The experimental test described in this deliverable will focus a common typology of monodirectional floors, composed by simple supported beams and transversal planking.

### 2.2 MECHANICAL ASPECTS

#### 2.2.1 Seismic behaviour of wooden floors

On the seismic behaviour of existent masonry constructions, the role of the floors is, first, to provide an effective connection for the walls subjected to perpendicular seismic actions, and second, to transfer and distribute the horizontal actions to the supporting walls arranged parallel to the seismic action.

#### 2.2.2 Importance of in-plane stiffness and strength (Horizontal Forces)

The main function of a floor in seismic areas is to transfer, through its strength and stiffness, the in-plane seismic forces that are created in the floor to the walls parallel to the seismic action. If the floor is deformable, it transfers the seismic action partly to the walls parallel to this action and partly to the orthogonal ones, Figure 2.4.

Another function of the floor is to distribute the seismic actions among the different supporting walls.

- If the floor is rigid and not deformable, the seismic forces are distribute among the different supporting walls parallel to the direction of the seismic action, in accordance with the walls stiffness, and considering the distance between the resultant of the in-plane forces and the centre of stiffness, Figure 2.5 (a).
- Instead, if the floor is deformable, the seismic forces are distributed among the different supporting walls parallel to the seismic action, in accordance with their influence areas regardless of the stiffness of each septum, Figure 2.5 (b).
2.2.3 Importance of the connection to the supporting walls

Finally, during a seismic event the floor acts as constraint on to the supporting walls, linking them together and ensuring a box type behaviour to the building.

This constrain reaction should be transferred from the floor to the walls parallel to the seismic action, soliciting the floor’s in-plane bending; to this purpose it’s however required a certain strength and stiffness of the floor to avoid the rupture of the walls orthogonal to the seismic action after excessive movements.

In the absence of an adequate connection between floor and walls, the seismic actions can overcome the friction forces and the beams may slip causing several vibrations modes between the various wall elements.

In what concerns the capacity of the floors to distribute the in-plane seismic actions among the various supporting walls, there are two types of buildings:

- buildings with the floors effectively connected to the masonry but without adequate in-plane resistance and stiffness (floor without reinforced concrete slab, or floor with simple or double planking);
- buildings with floors effectively connected to the masonry with suitable in-plane strength and stiffness (floors with reinforced concrete slab).

What is certainly desirable is to create effective connections between the walls and the horizontal structures in order to prevent the out-of-plane overturning of the walls, Figure 2.6.
2.2.4 Damages Symptoms

In this point it we will identify the manifestations of damage and degradation found in the elements that compose wooden floors: extrados, intrados, primary and secondary supporting frames. The biggest part of the degradation and damage possibilities that can be visually surveyed on wooden floors are manifested through:

i. the deformation of the bearing frame;
ii. the failure of the floor supports or through;
iii. material degradation.

2.3 AVAILABLE STRENGTHENING TECHNIQUES

2.3.1 Seismic improvement of wooden floors

As mentioned before, on the seismic behaviour of existent masonry constructions, the role of the floors is, first, to provide an effective connection for the walls subjected to perpendicular seismic actions, and second, to transfer and distribute the horizontal actions to the supporting walls arranged parallel to the seismic action. As so, the main problems affecting the structural behaviour of historical wooden floors are inevitably due to: (i) the lack of bending and in-plane stiffness, (ii) to the inadequate mechanical properties of the existing material and (iii) to inadequate connections between floors and supporting walls. In the following, current strengthening techniques for the improvement of the floor’s behaviour are presented in respect to the aforementioned problems.

2.3.1.1 Improvement of the in-plane stiffness

Several strengthening techniques have been researched and developed in order to increase the bending stiffness and load-bearing capacity of existing wooden floors, particularly in the case of simple frame floors made of rectangular load-bearing timbers, (Tampone, 1996). Next are summarized the main stiffening techniques currently used, that range from the execution of a reinforced concrete layer, to the use of steel ties, beams or plates, and reinforcement techniques according to the principles of sustainability, with the use of timber elements. The types of intervention are presented according to the type of material used.

i. Improvement interventions through the use of wood;
   • orthogonal or diagonal planking;
   • only wood technique - Timber flange connected by wooden dowels to main beams, (Modena et al., 1997).

ii. Improvement interventions through the use of steel elements;
   • metallic plates;
   • metallic diagonals;
• intervention using metallic plates and diagonals, (Gattesco et al., 2007).

iii. Improvement interventions through the use of a reinforced concrete cooperating slab;
• the Turrini-Piazza (1983) method;
• the Alessi, Lamborghini, Raffagli (1989) system;
• the Tampone (1992) system.

iv. Improvement interventions through the use fibro-reinforced composite materials;
• intervention using FRP, (Angotti et al., 2005);
• intervention using GFRP, (Borri et al., 2004).

2.3.1.2 Improvement the connection between floors and walls
The floor-wall connection is the most critical point of the buildings with deformable floors because the connection allows transferring the inertia forces from the floors to the walls and profoundly influences the walls out-of-plane behaviour. In the consolidation interventions of historical constructions a lot of attention is given to the connections in order to induce a box type behaviour to the structure. Next are enumerated different solutions to anchor the floor to the walls.

i. improvement through the insertion of ties;
ii. improvement through the insertion of pins;
iii. improvement through the insertion of bars;
iv. improvement through the insertion of metal plates;
v. improvement through the insertion of reinforced concrete beams;
vi. improvement through the connection between adjacent floors.

Some of the above mentioned interventions are more appropriate. For example the use of reinforce concrete elements, for the improvement of connections, produces negative consequences on the structural behaviour of the masonry structures. Moreover this intervention is not compatible with the conservation’s criteria.

2.4 AVAILABLE EXPERIMENTAL PROCEDURES
In literature several studies can be found on masonry buildings with deformable floors, only some of these focuses on the seismic response of the floor and on the influence of planking deformation in the seismic behaviour of walls. Many of these experimental works focus on buildings entirely made of wood or buildings in reinforced concrete which, due to its characteristics and mass involved, can sometimes lead to different conclusions. In the following some experiments confirming the theoretical behaviour described in the previous points are presented. Here are summarized examples of laboratory tests involving the creation of scale models of masonry buildings with flexible floors and tests on floors.

i. Dynamic tests (shaking table):
• Dynamic experimental tests (shacking table) performed on masonry buildings with different types of floors (Tomaževic, 1991).
• Seismic response of short masonry buildings with flexible floors (Cohen et al., 2001).
• Influence of the floors flexibility on the out-of-plane response of masonry walls (Sismir, 2004).

ii. Quasi-Static Tests:
• Experimental tests on masonry buildings and flexible floors (Paquette, 2003).
• Stiffening of a short building with flexible floors (Franklin, 2004).
• Seismic behaviour of timber floors in masonry buildings (Peralta et al, 2004).
2.5 REVIEW OF AVAILABLE MODELS AND DESIGN PROVISIONS

Are described next some modelling experiences of structures with deformable floors subjected to seismic loads: some concern modelling of existing buildings, others are numerical models developed based on laboratory experiments. Finally, some proposals of analytical modelling of structures with deformable floors are summarized. Tests are then divided into analytical models and numerical models. The modelling proposals summarized next focus mainly on the considerations concerning the floors deformation.

i. Analytic Modelling:
   - Seismic behaviour of two masonry structures with flexible floor (Abrams, 1994).
   - Linear analysis on short buildings (Kim, 2003).

ii. Numerical Modelling:
   - Linear elastic analysis using finite elements of short masonry buildings with flexible floors (Cohen et al., 2001).
   - Seismic linear analysis of high buildings considering the floor’s flexional stiffness (Lee, 2002).
   - Non-linear analysis on short buildings (Kim, 2004).
   - Tridimensional seismic analysis of a masonry building with flexible diaphragms (Sweeney et al., 2004).

2.6 CONCLUDING REMARKS

Due to the key role that horizontal elements play in the seismic behaviour of buildings, and their relatively unknown behaviour, there is a need for proper identification of parameters for the characterization of the in-plane stiffness of original wooden floors; which is crucial for the definition of design and assessment formulations. Also the actual improvement obtained by means of different strengthening solutions for the various construction typologies need to be classified in terms of resulting performance on reinforced structures. This characterization and validation can only be carried out by testing, on which, however, harmonized test procedures are still not available.
3 MASONRY VAULTS

3.1 INTRODUCTION

In the architecture world, arched structures can generate a projection, which is commonly known as a vault. Generally, they run in a horizontal direction. They can be seen anywhere from architecture built in ancient Egypt to architecture from the Gothic period in Europe.

From the Roman age to the Renaissance, the problem of stability and correct design of arches was mainly faced under a geometrical point of view (Figure 3.1), even though the presence of horizontal thrusts was already perceived at least in the 1st century B.C., as reported by Vitruvius within De Architectura, (Benvenuto, 1981). Wide reviews of the historical progress of studies on masonry arches and vaults can be found in Benvenuto (1981), Heyman (1982), Carbone et al. (2001).

![Figure 3.1 - Geometrical design rule still used during the 18th century, (Benvenuto, 1981).](image)

3.1.1 Typologies

There are different styles of vaults existing in building structures from ancient times to today. The vaulted structures can be typologically divided, according to its geometry, which is one of its most important parameters, essential to determine the proper structural behaviour. According to Carbonara in (2004) the vaults can be typologically classified as:

i. Translation Vaults, generated by the movement of a straight line (generatrix) along a curve:
   - Barrel vault, Figure 3.2 (a);
   - Groin vault, Figure 3.2 (b);
   - Barrel vault with lunettes, Figure 3.2 (c);
   - Cloister vault, Figure 3.2 (d);
   - "Volta a schifo", Figure 3.2 (e).
ii. Rotations Vaults, defined the rotation of a curve around an axis:

- Ribbed vault, Figure 3.3 (a);
- Dome, Figure 3.3 (b);
- Circular barrel vault, Figure 3.3 (c).
3.1.2 Structural behaviour

The vaults structural mechanism (similar to the arches) takes advantage of the natural crushing of the elements that compose it, provided that two basic conditions that ensure the equilibrium are guaranteed:

i. **Vaults stability**: a generic anti-funicular that balances the loads must be within the profile on every section causing a compression stress state compatible with the composing materials; eccentric compression would lead to unsustainable masonry bending (due to its almost absent tensile strength).

ii. **Global stability**: the piers on which the vault is supported should be able to accommodate the horizontal thrust generated by it.

These conditions, essential for the stability of the element, are the two principles that guide the design of a vaulted structure that function well, and in case of an existing building, that allow assessing the structural safety.

The "pushing" that characterizes these structures, makes them intrinsically prone to instability/damage, propensity that can be enhanced by any seismic phenomena.

3.1.3 Damage mechanisms and its causes

It is possible to classify the vaults collapse mechanisms based on its causes. Among the possible sources of damage it is possible to distinguish:

i. The relative displacement of the supports:
   a. displacement of the supports on the orthogonal direction to the generatrix line of the vaults;
   b. differential settlement of the piers;
   c. longitudinal sliding.

ii. The variation of the load to which the vaults and piers are subjected.

iii. The decay of masonry.

Figure 3.3 - Rotation vaults. (a) Ribbed vault, (Isawi, 2001). (b) Dome, (Isawi, 2001). (c) Circular barrel vault, (Carbonara, 2004).
3.1.4 Types of interventions

Various techniques, traditional or innovative, are available to counteract problems affecting masonry vaults, each of them presenting both positive and negative aspects. Next is presented a brief overview of the traditionally used types of consolidation according to Giardina (2006).

i. Buttress;
ii. Counter vaults in reinforced concrete;
iii. Counter vault in lime mortar and FRP net;
iv. Transversal vertical diaphragms, (frenelli);
v. Tie at the intrados;
vi. Tie at the extrados;
vii. Vertical tie;
viii. Curve tie;
ix. Suspension tie;
x. Cross ties at the extrados;
xii. Overcoat with composite material strips;
xiii. Reinforcement arches;
xiv. Lightweight spandrel.

In order to improve the resistance against horizontal thrusts, external buttresses (Figure 3.4) or steel ties, internally placed (more effective, Figure 3.5) or added over the extrados, may be provided.

The addition of masonry panels (called “frenelli” in Italy, Figure 3.6) or ribs (lightweight ones in Figure 3.7, Giuriani and Marini, 2008) on the extrados surface can improve the global behaviour in terms of stiffness and failure load.

Counter vaults in reinforced concrete were often used in the recent past, aimed at giving flexural strength to the masonry structure, Figure 3.8.

The application of FRP composite materials, which may involve the extrados or the intrados surface (Figure 3.9 and Figure 3.10), is becoming a widespread intervention, aimed at stitching the opening of hinges and giving flexural strength to the vault without increasing dead loads, differently from reinforced concrete counter vaults. In the case of intrados application, horizontal thrusts are supposed to be sensibly reduced, (Focacci, 2008).

Other techniques, such as the application of pre-stressed steel cables (Figure 3.11; Jurina, 1999) or steel textiles (Figure 3.12; Borri, 2006), have been proposed to the same purpose of improving the behaviour by stitching possible hinges.
3.1.5 Review of available models and design provisions

The analysis of dry block masonry structures by bound approach is usually restricted to cases in which sliding is prevented by high friction among block interfaces. This leads, for arches, to the well-known hinging mechanisms first discussed in terms of plastic analysis by Heyman (1966). However, especially for historic buildings the quality of the contact surfaces or of the binding materials might be deteriorated so as to substantially reduce the original friction coefficient. In addition, some particular shapes of curvilinear structures, e.g. flat arches, would never collapse unless sliding occurred. Hence it is necessary to study this group of problems under the more realistic assumptions of presence of sliding and absence of dilatancy. Casapulla and D’Ayala (2001) presented a proof of uniqueness of the solution for the limit state analysis of 3D masonry arches, in the condition of axial symmetry of geometry and loading. This proof is crucial to the robustness of the results and allows a straightforward treatment of this class of problems as one of standard limit-state analysis. The analysis can be easily extended to barrel vaults, common in historic buildings and forming the structure of masonry bridges. The same authors applied with success the method to domes (D’Ayala and Casapulla, 2001) and more recently Tomasoni and D’Ayala (2008) extended the procedure to the study of pavilion vaults.
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Internal forces at a general block interface of the arch
(Casapulla and D’Ayala, 2001).

Meridian stress resultant for unit length of parallel at each
interface

\[ S_j = \frac{\sum_{k=1}^{j}(W_k + P_k)}{\sin \beta_j} \]

and

\[ M'_X = M'_X - P \cdot y' \]

with

\[ N_j = S_j \cdot \cos(\alpha_j - \beta_j) \]

\[ T_j = S_j \cdot \sin(\alpha_j - \beta_j) \]

\[ M_j^t = M_j^t \cdot \cos \alpha_j \]

\[ M_j^t = M_j^t \cdot \sin \alpha_j \]

Meridian stress resultant after cracking

\[ S^* = \sqrt{(S_{j-1} \cos \alpha_j \cos \gamma_j)^2 + (S_{j-1} \cos \alpha_j \sin \gamma_j + W_j)^2} \]

corresponding angle:
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(a) View from the top of the pavilion vault, with the forces for a generic element. (b) Axonometric showing the angles $\gamma$ and $\alpha_k$ (D’Ayala and Tomasoni, 2008).

\[ \gamma^* = -\tan^{-1} \left( \frac{S_{j-1} \sin \gamma_{j-1} + W}{S_{j-1} \cos \gamma_{j-1}} \right) \]

and normal and shear resultant at the interface

\[ N_j = S^* \cdot \cos (\theta_j - \gamma_j^*) \]
\[ T_j = S^* \cdot \sin (\theta_j - \gamma_j^*) \quad \left| T_j \right| \leq T_0 + N_j \mu \]

where

\[ \alpha_k' = \sec^{-1} \left( \cos \gamma_j \cdot \sin \alpha_k \right) \]
\[ W_j = \frac{\omega R^2 (1 - \cos \theta_{j+1})}{n} - \frac{\omega R^2 (1 - \cos \theta_j)}{n} \]

$\omega$: weight for unit surface

$n$: number of meridian slices

\[ R' = \sqrt{\left( x_{\text{geom}} \right)^2 + \left( z'_j \right)^2} = \sqrt{\left( \frac{x_{\text{geom}}}{\cos \alpha_k} \right)^2 + \left( z'_j \right)^2} \]
4 ROOFS

4.1 INTRODUCTION

The roof structure is usually composed by timber beams or trusses. The roof structure could thrust directly the walls or be simply supported as in case of trusses, (Figure 4.1). In some case, wooden tie rings mitigated the thrust from the roof, and constrains the walls on top.

Traditional roof structures include several configurations, spanning from about 5.0 to over then 25.0 m, according to complexity of the structure, (Figure 4.2). The configuration of the secondary structure supporting the covering influences the truss distance, (Figure 4.3). Wooden structures could have further configuration, as wooden vaults, (Figure 4.4).

Figure 4.1 - Classification of the roof structures diffuse in Italy according their thrust; a) Thrusting structure; b) Reduced thrust; c) not thrusting. From the post-earthquake damage survey form (GNDT, 1999), (Aedes, 2000).

Figure 4.2 - Example of roof trusses from (Rondelet, 1834).

Figure 4.3 - Example of roof covering structure, (Giordano, 1999).
4.1.1 Problems under seismic action

Roof damage could be revealed by a movement of the joints or of the ridge (Figure 4.5).

Local damages caused by the movement of the tile coverings are frequently causes of the beams decay. In Figure 4.6 and Figure 4.7 the decay of the connection between strut and tie causes a sliding of the strut which thrust directly the wall. Strut-tie decay is frequently surveyed due to the biological attack or moisture conditions, (Figure 4.8). The damage could involve the truss collapse or an anomalous loading of the supporting walls. Thrusting elements could contribute to the local or global overturning of unrestrained walls, (Figure 4.9 and Figure 4.10).
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Figure 4.6 - The ridge movement produces the struts pushing on the side walls, (Doglioni, 2007).

Figure 4.7 - Thrust to the load bearing walls caused by truss damages, (Tampone, 2001).

Figure 4.8 - Truss collapse due to the decay of the strut-tie node, (Doglioni, 2007).

Figure 4.9 - Roof thrust damage on a multiple leaf stone masonry, (Doglioni, 1999).

Figure 4.10 - Hammering of the roof structures to the load bearing walls, (Doglioni, 1999).
4.2 Experimental campaigns

4.2.1 Testing on full-size timber trusses (Parisi et al., 2007)

Two full-scale timber trusses belonging to the Teatro Sociale di Trento, in northern Italy, were dismantled and donated to the University of Trento, where a research program on structural timber was in progress, (Parisi et al. 2007) (Parisi and Piazza, 2003), Figure 4.11. The laboratory for structural and material testing of the University of Trento was sufficiently large to accommodate the over 25.0 m span and 6.4 m height of the trusses.

The trusses were subjected to various rehabilitation interventions, remounted and laboratory tested. Interventions included reintegration of chord ends and rafters, connections restoring continuity in the chords, and reinforcement at the chord toe, where the rafter is inserted, by a distribution of steel screws. This operation had shown in previous tests to highly reduce the weakness in shear of the region, (Parisi et al., 2000).

The research project intended to:

i. analyze and test the different aspects of the restoration of timber structures, ranging from historical documentation to reconstruction techniques;
ii. apply different methodologies of analysis, from experimentation to numerical modelling and evaluation.

An extended series of non-destructive tests were performed on the trusses:

i. Inspection for the visual strength grading.
ii. More than 900 measurements of the moisture content.
iii. About 2000 sclerometer (Pilodyn) tests.
iv. About 1600 hardness tests.
v. About 100 measurements of the microdrilling resistance
vi. Resistograph.
vii. 60 dynamic tests with instrumented hammer.
viii. 180 ultrasonic (Sylvatest) tests.

The mechanical characteristics, such as the modulus of elasticity, were derived through these non-destructive tests employing a variety of methods.

4.2.2 Testing on timber joints (Parisi et al., 2000)

The experimental research was carried out at the Structural Testing Laboratory of the University of Trento, and consisted of monotonic and cyclic tests of full-size joints, (Parisi and Piazza, 1998),
Figure 4.12. Tests on unreinforced joints were performed in order to acquire information on the primary behavioural characteristics of the connection as well as its sensitivity to a few parameters. A refined quantification of the effects of other parameters was done using a numerical analysis. Monotonic loading tests were performed first on a set of unreinforced assemblies. Subsequently, two sets of joints that were reinforced with basic devices, Figure 4.13, were tested with monotonic and cyclic loads. The purpose of these tests was to uncover any advantages and deficiencies in the behaviour of the joint and of the device itself, as well as to determine a need for different types of reinforcement.

![Figure 4.12 - Test on Birdsmouth Joint: Testing Apparatus, Instrumentation Layout, Positive and Negative Force Direction, (Parisi et al., 2000).](image)

Figure 4.13 - Different Types of Traditional Strengthening Techniques for Birdsmouth Joint, (Parisi et al., 2000).

4.2.3 Testing on joints (Branco et al., 2006)

A series of tests on unstrengthened specimens were performed in order to characterize the original behaviour of joints representative of existing timber systems, Figure 4.14. Subsequently, a set of joints were strengthened with metal devices and tested under monotonic and cyclic loading, Figure 4.15. Tests on assembled connections were preceded by accurate material characterization, in terms of the mechanical properties of the timber elements used for all full-scale models.

![Figure 4.14 - Experimental test setup, (Branco et al., 2006).](image)
4.2.4 Experimental tests on Timber trusses, (Rodrigues, 2004)

Some structural elements of the wooden structure of the roof belonging to the “Salão de Festas” of the Arouca Monastery, namely two wooden trusses, Figure 4.16, were tested with the objective of studying its unstrengthened and strengthened resistant capacity.

The two timber trusses were tested until collapse, obtaining this way the maximum resistant strength applied to the top of the king post as illustrated in Figure 4.17.
The trusses were then reinforced according to the specifications in Figure 4.18 and retested using the previously presented test setup.

![Figure 4.17 - Experimental test setup, (Rodrigues, 2004).](image)

![Figure 4.18 - Strengthening solutions applied to the trusses, (Rodrigues, 2004). (a)Truss 1. (b) Truss 2.](image)

**4.2.5 Experimental tests on dovetail joint for timber roof, (Garbin et al., 2006)**

The dovetail joints are within carpentry joints, as the previous they were expensive to make, but they have become economically advantageous in the last years because the progress on machine tools applied on timber manufacturing. This joint typology is characterized by a trapezium shaped cross section and it is able to transmit axial and shear loads. Those considered in this contribution are used to connect beams for timber roofs and they are mainly subject to shear loads.
To study this type of connection it were carried out shear tests on the joints. Several variables were taken into account in order to characterize the connection, in particular: the angle of the tail, the wood fibre configuration of the tenons and the geometry of the groove. Different failure modes were observed depending on the groove geometry (Figure 4.20 and Figure 4.21).

Based on the experimental results (Figure 4.22) an empirical model was developed in order to describe and design the behaviour of the dovetail joints.
4.3 Reinforcement techniques

The methods for strengthening timber elements, connections, and structures depend on the type of problem, on the element, and on the objective to be reached. Yet, all are generally based on the collaboration between timber and other materials, as metals, fibers, or concrete. The intervention methods may be classified into two basic groups. A first one gathers various types of interventions based on the addition of material embedded in the wood, supplying the mechanical characteristics that are missing in the original material or element, either because of degradation during its lifetime, or because of inadequate original design. A second, and at times alternative, methodology is to put in parallel to the original element or system new collaborating elements, giving place to a mixed or composite system. The critical point, particularly for embedded systems, is the realization of adequate connections.

4.3.1 Connections reconstituting elements

The connection techniques between timber elements can be classified in two main categories such as mechanical joints or carpentry joints. The first refer to connections made by steel elements sometime glued with suitable resins, the seconds refer to traditional workmanships often used in the past to connect timber structures.

The traditional connection systems, both for ancient and modern wooden structures, consist of carpentry joints, transmitting forces directly between elements, or by mechanical joints, in which load transfer is performed through special devices, usually metal connectors.

Some of the most interesting new types of connections, that have been proposed for possible future development, are those utilizing glued-in steel elements (bars or plates), completely hidden inside the timber. Many examples of these connections are now available in the field of modern constructions in glulam: examples are reported in Natterer, Herzog and Volz, 1994. Interesting applications may be found also in the field of structural repair and rehabilitation (Piazza, 1994; Piazza et al., 1999).

- The basic connections with glued-in steel elements
- Glued-in parallel-to-the-grain steel bars subjected to tensile forces
- Glued-in perpendicular-to-the-grain steel bars subjected to shear forces

Usually, carpentry joints, which transmit forces by contact between the timber elements relying on compression and friction, are reinforced with metal parts in order to avoid separation under exceptional actions and possibly to increase the mechanical properties of the connection.

The most common joint in existing roof timber structures is the “birdssmouth joint with a single tooth”, although geometry varies with joint location in the truss, and the joint bearing capacity is function of skew angle, notch depth and length of the toe.
Joints strengthening can be done in a number of possible ways: from simple replacement or addition of fasteners, to the use of metal plates, glued composites or even full injection with fluid adhesives. Each solution has unique consequences in terms of the joint final strength, stiffness and ductility. Although being widely used, the number of studies on the mechanical performance of existing traditional carpentry joints and possible strengthening techniques is not worldwide.
5 PLANNING OF THE EXPERIMENTAL RESEARCH

5.1 WOODEN FLOORS

It is clear from previous sections that identification of parameters for the characterization of the in-plane stiffness of timber floors is crucial to define design and assessment formulations. Harmonized test procedures are not available, and the large variability of strengthening solutions for the various construction typologies, need to be classified in terms of resulting performance on reinforced structures. To this scope a comprehensive experimental work is proposed. Tests are distributed among partners as follows.

5.1.1 Partner n° 1 and 13 - UNIPD and BOZZA

5.1.1.1 Objectives

UNIPD will analyze and characterize the effect of different strengthening techniques on the in-plane stiffness of timber floors. The experimental program has been defined in strict collaboration with BOZZA Legnami S.r.l., to pursue a proper balance among scientific issues and current market demand. BOZZA will provide materials and expertise for realization of samples, that will be tested and analyzed by UNIPD. Diagonal tests (monotonic and cyclic) on full scale timber floor elements will be performed, in both unreinforced and reinforced conditions. These strengthening solutions will be executed using both traditional (orthogonal and diagonal planking, diagonal punched steel strips, wooden diagonal struts) and innovative materials, such as Fibre Reinforced Polymers (FRP) using Carbon, Basalt, Steel or Natural long fibres (also known as CFRP, SRP and Green Composites), and gypsum fibre boards. Table 5.1 lists the tests on strengthened floors that will be carried out at UNIPD.
Table 5.1 - Test matrix for shear tests (Monotonic and Cyclic) on strengthened floors.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of test</th>
<th>Original planking</th>
<th>Strengthening n° 1</th>
<th>Strengthening n° 2</th>
<th>Layout</th>
</tr>
</thead>
<tbody>
<tr>
<td>FMSB</td>
<td>Monotonic</td>
<td>Simple planking thickness 20.0mm</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>FM</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>FC</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>FMSD</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Diagonal punched steel strip at +45.0°</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>Specimen</td>
<td>Type of test</td>
<td>Original planking</td>
<td>Strengthening n° 1</td>
<td>Strengthening n° 2</td>
<td>Layout</td>
</tr>
<tr>
<td>----------</td>
<td>--------------</td>
<td>-------------------</td>
<td>--------------------</td>
<td>--------------------</td>
<td>--------</td>
</tr>
<tr>
<td>FCSD</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Diagonal punched steel strip at +45.0°</td>
<td>Diagonal punched steel strip at -45.0°</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>FM+45°SP(A)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Simple planking thickness 25.0mm at +45.0°</td>
<td>---</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>FC+45°SP(A)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Simple planking thickness 25.0mm at +45.0°</td>
<td>---</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>FM+45°SP(B)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Simple planking thickness 40.0mm at +45.0°</td>
<td>---</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>FC+45°SP(B)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Simple planking thickness 40.0mm at +45.0°</td>
<td>---</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>FM+45°SP(C)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Tongue and groove planking, thickness 33.0mm at +45.0°</td>
<td>---</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Specimen</td>
<td>Type of test</td>
<td>Original planking</td>
<td>Strengthening n° 1</td>
<td>Strengthening n° 2</td>
<td>Layout</td>
</tr>
<tr>
<td>-----------</td>
<td>--------------</td>
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<td>--------------------</td>
<td>--------</td>
</tr>
<tr>
<td>FC+45°SP(C)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Tongue and groove planking, thickness 33.0mm at +45.0°</td>
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<tr>
<td>FM±45°DP(A)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Tongue and groove planking, thickness 25.0mm at +45.0°</td>
<td>Tongue and groove planking, thickness 25.0mm at -45.0°</td>
<td><img src="image2" alt="Diagram" /></td>
</tr>
<tr>
<td>FC±45°DP(A)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Tongue and groove planking, thickness 25.0mm at +45.0°</td>
<td>Tongue and groove planking, thickness 25.0mm at -45.0°</td>
<td><img src="image3" alt="Diagram" /></td>
</tr>
<tr>
<td>FMWD(D)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Wooden diagonal thickness 25.0mm at +45.0°</td>
<td>---</td>
<td><img src="image4" alt="Diagram" /></td>
</tr>
<tr>
<td>FMWD(E)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Wooden diagonal thickness 50.0mm at +45.0°</td>
<td>Wooden diagonal thickness 50.0mm at -45.0°</td>
<td><img src="image5" alt="Diagram" /></td>
</tr>
<tr>
<td>FCWD(E)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Wooden diagonal thickness 50.0mm at +45.0°</td>
<td>Wooden diagonal thickness 50.0mm at -45.0°</td>
<td><img src="image6" alt="Diagram" /></td>
</tr>
<tr>
<td>Specimen</td>
<td>Type of test</td>
<td>Original planking</td>
<td>Strengthening n° 1</td>
<td>Strengthening n° 2</td>
<td>Layout</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>--------</td>
</tr>
<tr>
<td>FM FRP</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20mm</td>
<td>Carbon, Basalt or Steel Fibers disposed diagonally, at +45.0°</td>
<td>Carbon, Basalt or Steel Natural Fibers disposed diagonally, at -45.0°</td>
<td><img src="image1.png" alt="Diagram" /></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td><img src="image2.png" alt="Diagram" /></td>
</tr>
<tr>
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<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Carbon, Basalt or Steel Fibers disposed diagonally, at +45.0°</td>
<td>Carbon, Basalt or Steel Fibers disposed diagonally, at -45.0°</td>
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<td><img src="image4.png" alt="Diagram" /></td>
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<tr>
<td>FM GC</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20mm</td>
<td>Natural Fibers disposed diagonally, at +45.0°</td>
<td>Natural Fibers disposed diagonally, at -45.0°</td>
<td><img src="image5.png" alt="Diagram" /></td>
</tr>
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<td></td>
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<td></td>
<td><img src="image6.png" alt="Diagram" /></td>
</tr>
<tr>
<td>FC GC</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Natural Fibers disposed diagonally, at +45.0°</td>
<td>Natural Fibers disposed diagonally, at -45.0°</td>
<td><img src="image7.png" alt="Diagram" /></td>
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</tr>
<tr>
<td>FM FRP +GFB (A)</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Carbon, Basalt, Steel or Natural Fibers disposed diagonally, at +45.0° + Carbon, Basalt, Steel or Natural Fibers disposed diagonally, at -45°</td>
<td>Upper GFB, thickness 18 mm to avoid FRP buckling,</td>
<td><img src="image9.png" alt="Diagram" /></td>
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</tbody>
</table>

Optimization of design for floors, roof and vaults

D5.1 29
## NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

### Optimization of design for floors, roof and vaults

#### Specimen Table

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of test</th>
<th>Original planking</th>
<th>Strengthening n° 1</th>
<th>Strengthening n° 2</th>
<th>Layout</th>
</tr>
</thead>
<tbody>
<tr>
<td>FC FRP +GFB (A)</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Carbon, Basalt, Steel or Natural Fibers disposed diagonally, at +45°0 + Carbon, Basalt, Steel or Natural Fibers disposed diagonally, at -45°</td>
<td>Upper GFB, thickness 18 mm to avoid FRP buckling.</td>
<td></td>
</tr>
<tr>
<td>FM GSB</td>
<td>Monotonic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Upper GFB, thickness 18 mm</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>FC GSB</td>
<td>Cyclic</td>
<td>Tongue and groove planking, thickness 20.0mm</td>
<td>Upper GFB, thickness 18 mm</td>
<td>---</td>
<td></td>
</tr>
</tbody>
</table>

### Legend:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>Floor</td>
<td>FRP Fibre reinforced polymers</td>
</tr>
<tr>
<td>M</td>
<td>Monotonic Test</td>
<td>GC Green Composites</td>
</tr>
<tr>
<td>C</td>
<td>Cyclic Test</td>
<td>GFB Gypsum fibre boards</td>
</tr>
<tr>
<td>SB</td>
<td>Simple boarding</td>
<td>(A) Planking thickness 25 mm</td>
</tr>
<tr>
<td>SD</td>
<td>Steel Diagonal</td>
<td>(B) Planking thickness 40 mm</td>
</tr>
<tr>
<td>WD</td>
<td>Wood Diagonal</td>
<td>(C) Planking thickness 33 mm</td>
</tr>
<tr>
<td>SP</td>
<td>Single Planking</td>
<td>(D) Diagonal thickness 25 mm</td>
</tr>
<tr>
<td>DP</td>
<td>Double Planking</td>
<td>(E) Diagonal thickness 50 mm</td>
</tr>
</tbody>
</table>
5.1.1.2 Test setup description

For the real scale floor specimens, a specific testing machine, that was designed and realized to adequately simulate the monotonic and cyclic shear behaviour of floors, will be used. This machine is capable of minimizing many undesired effects and the dimensional limitations.

Starting from the typical advantages of the vertical diagonal test, the proposed system consists in a vertical shear-bending test similar to the configurations used for testing woodframe shearwalls, (Gatto, 2003). The structural scheme provides three simple supports applied at the floor base, (Figure 2.1); it is equivalent to a cantilever beam, which can apply bending and shear deformations, as set in the Timoshenko beam theory (1934). Thanks to this structural scheme, the geometrical dimensions of the floor specimens could be almost half of those used in Peralta et al. (2004), and about a quarter of those usually present in existing masonry houses.

The steel articulated quadrilateral was designed such that out-of-plane motion at the specimen boundaries (bottom, lateral and top) is prevented, as it happens in buildings. On the contrary, it allows uplift and in-plane deformations of the sample, by minimizing frictional effects. Floor specimens are inserted in the steel frame with the beams placed horizontally, thus allowing the free rotation of the planking.

The bottom joist of the tested specimens will be fully fixed to the bottom steel reaction beam by means of bolts. The steel reaction beam is connected to the steel basement trough three load cells by means of mechanical sliding connections, thus providing the three simple supports. In the horizontal direction, a hydraulic actuator was used, to load the specimens at the level of the top beam. Tests will be performed in displacement control, and the force applied at the top of the samples is measured through a load cell. Finally, horizontal and vertical load cells apply the shear and the bending moment to the floor specimen in its plane.

A suitable instrumentation will be placed on both sides of each specimen, to capture the global and local shear-bending behaviour.

5.1.1.3 Floor specimens

The unreinforced tests specimens will be composed by timber floor elements (2.2×2.2 m), built with components in real size. The samples will reproduce a common typology of mono-directional floors, composed by simple supported beams and a transversal planking. In particular, five beams 12.0×14.0 cm in section, and a basic boarding 13.5 cm wide and 2.0 cm thick, will be used (Figure 5.2). Spruce wood will be used for the construction of all the elements. Each board will be joined to every beam by means of 2 nails Ø 2.75x60.0 mm, for a total of 32 nails for each beam and 10 nails for each board. Some specimen will be made with common raw-finished planking, whereas others will be constructed has a tongue-and-groove shaped connection in the planking thickness, (Figure 5.3).
5.1.2 Partner n° 3 - ITAM

5.1.2.1 Introduction

Many buildings (which are now considered heritage buildings) in Czech lands and adjacent regions have typically two types of wooden roofs. One, more traditional is made as a heavy timber floor (called "dübel" floors) which is composed of beams placed side by side closely together and fixed with dowels. It is a heavy, fire resistant floor with an infill and floor surface made typically from wood planks, see Figure 5.3. The second type is classical beam floors with an infill layer and a floor surface, see Figure 5.4 and Figure 5.5.

Timber ceiling is the eldest known type of the ceiling for any type of buildings. It has been used in medieval time for all social levels with maximum span up to 6m. It creates a logical complementary part to the log walls used in old wooden buildings. It is documented also, that there existed also a vaulted variant of this type of ceiling, (Škabrada Jiří, 2005).

Further development went into the direction of usage of ceiling beams beam floors with higher span. It has been used in the region of central Europe till mid of 20th century.

5.1.2.2 Objectives and methods

The authentic floor segments will be taken from the historic objects when available, or the true dummy will be build by historic masonry and structures experts. Then, they will be tested at the Central Laboratory of Experimental Mechanics at ITAM. The objective is to understand and describe quantitatively the behaviour of the floor under cyclic in-plane shear loading and subsequently to introduce as low as possible intrusive reinforcing. Also various floor surfaces will be used to compare their efficiency.

Series of in-plane loads of increasing displacement will be applied at the floor plane up to a level of displacement beyond the maximum load. The EN prescription will be followed. Displacement...
transducers and a load cell will measure the ceiling and floor global and local deformation and the applied load. The results will be compared to evaluate the efficiency of retrofit techniques. Finally, the results will be used to validate the numerical models for the simulation of timber and beam ceilings in a next step of the research.

Figure 5.4 - Timber ceiling without floor layer.
NEW INTEGRATED KNOWLEDGE BASED APPROACHES TO THE PROTECTION OF CULTURAL HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n° 244123

Figure 5.5 - Wooden floor with beams with tentative suspension used for experiments.

CEILING-COMPOSITION
- WOODEN FLOORS, ACCORDING TO THE TYPE OF TEST
- CLAY, 50 mm
- WOODEN FLOORS, 18 mm

Figure 5.6 - Detail of the tentative suspension.
5.1.2.3 Experimental programme
Experiments on authentic segments of wooden floor.
The study will consist on the experimental campaign using wooden ceiling. The experimental campaign will consist of:

- cycling loading of applied at the floor plane;
- comparison with numerical models, identification.

5.1.3 Partner n° 6 - UMINHO

5.1.3.1 Introduction
Most of the buildings built till the 40s-50s in Portugal is characterized by a structure made of masonry, mainly stone (exterior walls and some interior walls), and timber elements (floors, ceilings roofs, interior walls and some exterior walls). If well connected, these elements promote a good over whole behaviour: the masonry walls supports the floor beams and roof trusses which, if properly connected to the masonry, act as horizontal braces, inducing a more uniform behaviour and a better global performance. For instance, the out-of-plane deformation of a wall does not occur separately, but involves other walls through the floors and trusses in-plane stiffness. Thus, if properly designed and in good conditions, such constructions constitute efficient structures.

5.1.3.2 Objectives
The floor beams will be tested at the Laboratory for Structures (LEST) at UMINHO: isolated, following the standard procedures for beams under flexural moments, and integrated in a floor system, made of a series of beams and floorboards for shear tests. The floor beams will be tested up to rupture to assess the stiffness, strength and failure modes. Then, the beams will be retrofitted using traditional materials and tested again under the same setup and load conditions. The results will be compared to evaluate the efficiency of the techniques.

Floor structures will be constructed at LEST using timber beams and floorboards from old buildings. They will be shear tested to assess the in-plane stiffness of the floor, a characteristic with an important role on the behaviour of traditional masonry structures under seismic actions. Then, the floor will be strengthened using low invasive procedures, such as the addition of cross cables underneath the floor, or diagonal wood braces between beams.

5.1.3.3 Task description
This task aims assessing experimentally the mechanical behaviour and characteristics of timber floor beams under cyclic flexural moments, and of timber floor structures, constructed through the assemblage of beams and floorboards, under in-plane shear cyclic displacements. The beams and the floorboards will be retrieved from existing buildings; some of these elements (6 beams) are already at the UMinho, and contacts with private owners ensure the possibility of retrieving other specimens. Both structures will be tested at the UMinho under displacement control.

Before the load-carrying tests, a preliminary survey will be performed on the timber floors elements aiming to collect information on the mechanical and physical characteristics of the floor components, their state of conservation and connections to the bearing walls. Actually, this survey follows the activity of the team project in some of their everyday actions and provides useful information for the appraisal of existing structures.

The tests will be carried out according to EN408. A series of cycles of increasing displacement will be applied to the floor beams up to a level of displacement beyond the maximum peak load. Displacement transducers and a load cell will measure the beam’s global and local deformation and the applied load. The floor beams will be tested first in their actual state to assess the original mechanical characteristics, stiffness, strength and failure modes. Afterwards, according to the rupture mode, different repair and (or) strengthening techniques using traditional materials will be selected and applied to, namely the link of new wood elements to the existing ones through wood dowels, screws and (or) traditional joints, and the use of internal or external steel...
connectors/plates/belts. Then, the beams will be tested again, following the same setup and loading conditions of the first test. The comparison of the results before and after retrofitting, and between retrofitted beams, will give a good measure of the techniques efficiency.

Two floor structures will be constructed and tested at the LEST using 4 timber beams each and floorboards. The floor will be framed by a steel structure hinged at the corners and tested under in-plane shear forces applied to in the perpendicular direction of the beams. The structure will simulate half span of a 6 m span floor, meaning beams with 2 different links at the extremities: hinged along the anti-symmetrical line and having a special built in link type at the other extremity to simulate the connections to the wall. The floors stiffness and energy dissipation, characteristics with an important role on the behaviour of traditional masonry structures under horizontal actions, namely seismic, will be evaluated. The tests will be performed imposing cyclic shear displacements with increasing amplitude, first on the “original” floor and then on the strengthened floor using low invasive procedures, such as the addition of cross cables underneath the floor or diagonal wood braces between beams.

5.1.3.4 Expected results

This task will assess the mechanical characteristics and behaviour of real floor beams and floor structures before and after applying repair and (or) strengthening techniques, in particular: the bending stiffness, strength and rupture modes of the old floor beams, the in-plane shear stiffness of the floor structures and the energy dissipation of both structures. It will show that both beams and floors can be retrofitted using traditional materials, sustaining the no need for replacement under partial damage conditions. The task will point out possible rupture modes and will highlight the importance of the floors as horizontal bracing elements on traditional stone masonry constructions. Finally, the results will be used to validate the numerical models for the simulation of timber floors in a next step of the research.

5.2 MASONRY VAULTS

As previously highlighted, the adoption of a test setup involving only vertical loads may be considered adequate to investigate the limit capacity of arched structures and to calibrate and possibly improve available models.

5.2.1 Partner n° 1 - UNIPD

UNIPD will test eight barrel vaults made of clay brick masonry (Figure 5.7), adopting the same span, rise and width for each specimen (about 3.3 m, 1.1 m and 0.8 m, respectively), and symmetrically loaded by two alternative vertical line forces.

The investigated variables are: (i) thickness of the vault (t); (ii) reinforcement application (extrados or intrados); (iii) type of fibres and (iv) type of test (monotonic or cyclic). The planned combination of these parameters is listed in Table 5.2.

Two unreinforced specimens will be tested in order to compare the effects of the applied reinforcements in what concerns their effectiveness and the change of global behaviour (stiffness, ductility, energy dissipation, failure mechanisms).
Figure 5.7 - Test scheme for barrel vaults.

Table 5.2 - Test Matrix for tests on masonry barrel vaults.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vault thickness</th>
<th>Reinforcement application</th>
<th>Fibres type</th>
<th>Test type</th>
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<td></td>
<td>$t_1$</td>
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Additional testing will be performed, accordantly with its feasibility in terms of time and funds cost, on a one- or two-bay specimen, in order to investigate the behaviour of the vaulted system subjected to differential in-plane displacements of the abutments.

5.2.2 Partner n° 4 - NTUA

5.2.2.1 Experimental programme

The experimental campaign will consist on testing a subassemblage on an earthquake simulator. This tests will allow to check methods of improving the behaviour (and, hence, the effect) of "horizontal", elements to the seismic behaviour of buildings.

The subassemblage consists of a cross vault, resting on two parallel piers. The subassemblage will be tested perpendicular to the longitudinal axis of the piers. We are discussing also about the possibility of applying also vertical acceleration. This subassemblage will be strengthened using...
steel ties in the cross vault and FRPs (in vertical straps) to the piers (to enhance their flexural capacity).

This experimental test will be connected to WP7 (systemic improvement of overall seismic response).

5.2.3 Partner n°8 - UBATH

5.2.3.1 Background

As already observed in previous paragraphs, the primary criteria for safe behaviour of masonry structures are: strength, stiffness, and stability. However, the high stiffness of masonry makes stress-induced deflection irrelevant, whereas mean stresses tend to be extremely low (Heyman 1995).

Therefore the principal failure mode is ‘instability’, occurring as mechanism formation when tension cracking and stress localisation cause sufficient hinges to form within the structure. Small displacements in the supports can trigger the formation of such kinematic mechanisms at even lower loads, proving highly critical to uncracked masonry (Heyman 1995).

Unfortunately, the age of masonry buildings results in the tendency for masonry spanning structures to show signs of movement and deformation as consequence of one or a combination of: foundation subsidence, ground swell, construction defects, accidental impact, creep in the mortar and vibrations (Ochsendorf 2006). Of course, seismic events also contribute to cause movements of supports.

Conversely, a large increase in load capacity could be achieved by precluding mechanism formation as this is the lowest capacity failure mode. To this purpose the use of reinforcing material to carry tension loads has been suggested and programs of physical laboratory testing by Valluzzi et al. (2001) and Foraboschi (2004) have confirmed the validity of this technique in preventing the formation of hinges.

A notable application for FRP reinforcement has been the upgrading of masonry arch bridges. Modern traffic loads have markedly increased in respect to the conditions under which the majority of masonry bridges were built to operate, causing a pressing need to provide to the strengthening of arches, which experience the worst case condition of an eccentric point (axle) load applied at the quarter span.

Research into FRP reinforced masonry spanning structures has therefore been focused on these structures (Valluzzi et al., 2001; Foraboschi, 2004) and their two dimensional simplification, the circular arc voussoir arch (Boorer, 2005; Basilio et al., 2004). Although experimental testing of other reinforced vault structures has been reported by Foraboschi (2004), the mathematical models formulated in the same paper apply only to voussoir arches and barrel vaults.

Thompson (2007) investigated the effects of support movement on reinforced arches and barrel vaults by Discrete Finite Element Models (DFEMs), providing strong evidence that reinforcing arches with FRP can have a considerable effect on their sensitivity to support movement. For intrados reinforcement, DFEMs indicated that FRP failure might occur for very small support movements. This may raise concerns for the durability of existing interventions, since reported phenomena of support spreading in masonry structures showed typical displacements that greatly exceed those shown to cause serviceability failures in Thompson’s analytical models.

In intrados reinforced arches, both eccentric loads and support spreading appear to induce potentially coincidental moments in the arch. It is therefore possible that with high stresses present in the FRP due to support movement, a small increase in eccentric load could cause the capacity of the FRP to be exceeded (Figure 5.8). Once rupture has occurred, mechanism formation is no longer inhibited by the reinforcement, potentially reducing the load factor below that which the reinforced arch had been designed for.
This behaviour may have implications for the reinforcement strategy of heritage structures. FRP interventions tend to be used in structures of reasonable status or importance and receive ongoing inspection. As such, if support movements were to occur slowly over time, they might reasonably be noticed, investigated and remedial work undertaken before reaching these values. For these structures, increasing the arch’s ability to sustain large displacements is not critical. By contrast, in structures prone to seismic conditions, large movements may occur over a few seconds and, consequently, increasing the amount of support movement a structure can sustain by perhaps 300% might be considered an excellent way of increasing its robustness and safety under seismic action.

Thompson’s study underpins the distinction between strengthening a structure and reducing its potential for collapse. Reinforcing an arch with FRP at its intrados dramatically increases its strength and resistance to initial spread but, significantly, may have the potential to decrease its capacity for sustaining movement of its abutments. By contrast, whilst offering no additional resistance to the initial spread, extrados reinforcement can dramatically increase the ability of the arch to sustain support displacement, arguably producing a more robust structure.

Thompson’s FE simulations suggest that the presence of FRP at the arch extrados prevents haunch hinges opening and therefore prevents the collapse mechanism found in unreinforced or intrados reinforced arches whereby a central portion of the arch rotates around haunch hinges and collapses. The principal failure mode is by rotation of two composite members about abutment hinges. This indicates that the prediction of support displacement required to cause collapse could arise from purely geometrical considerations.

Following such approach, Thompson proposes the following solution for the support displacement, \( D_{ult} \), required to cause the collapse for an arch reinforced with FRP at the extrados:

\[
X = \sqrt{h^2 + \frac{L^2}{4}}
\]

\[
\theta = \tan^{-1}\left(\frac{2h + 2t}{L}\right)
\]
\[ \vartheta_1 = \tan^{-1}\left( \frac{2h + 2t}{L} \right) - \tan^{-1}\left( \frac{2h}{L} \right) \]

\[ \vartheta_2 = \tan^{-1}\left( \frac{2h}{L} \right) \]

\[ Y_1 = X^2 - x^2 \]
\[ Y_2 = t^2 - x^2 \]
\[ x = X \sin \vartheta_1 \]

Letting \( \varphi = h^2 + \frac{L}{4} \)

\[ \phi = \sin^2\left( \tan^{-1}\left( \frac{2h + 2t}{L} \right) - \tan^{-1}\left( \frac{2h}{L} \right) \right) \]

and

\[ Y_1 = \sqrt{\phi (1 - \varphi)} \quad \text{and} \quad Y_2 = \sqrt{t^2 - \phi \varphi} \]

\[ Y_T = \sqrt{\phi (1 - \varphi) + \sqrt{t^2 - \phi \varphi}} \]

When \( D > (2YT-L) \), collapse occurs, hence \( D_{ult} = 2YT-L \).

Thompson’s study, however, was limited to a set of initial hypothesis and further literature on this specific topic is missing so that more comprehensive studies would be needed to draw some final conclusions.

5.2.3.2 Objectives

A large proportion of historic masonry bridges in Europe are located in conservation areas and/or are listed structures and in fact their preservation and strengthening constitute a problem, especially considering that historic bridges are still used and are considerably overloaded because of nowadays road and railway traffic.

Repair is often performed using hydraulic lime, which is advantageous for arches built over rivers with large variations in tide level as the mortar dries out even if wet. Furthermore, being lime mortar used in the original construction, its use allows a ‘like for like’ replacement, which is indeed required for listed buildings.

In the UK, bridge assessment is performed according to British standards such as BD21, various BA codes and MEXE analysis. These procedures are suitable for assessing relatively modern bridges that can be retrospectively repaired/ strengthened with relative ease.

However, data contained in the aforementioned documents are obtained by static load tests undertaken by various institutes, such as the Transport Research Laboratory (TRL), using of engineering bricks and cementitious mortar so that samples are not representative of real cases.

Therefore UBATH built two set of three masonry voussoir arches.

i. For the monotonic tests, following Thompson’s (2007) computational analysis and his theoretical conclusions described in section 5.2.3.1, UBATH investigate and compare the collapse mechanism and ultimate displacement of three arches with:

- no reinforcement (Arch 1);
- SRP sheet anchored at the intrados (Arch 2);
- SRP sheet anchored at the extrados (Arch 3).

i. For dynamic tests UBATH built a set of three masonry voussoir arches (Lopata, 2006) to:

- quantify the deterioration in masonry strength during repeated load cycles;
- produce more realistic test data that can be used to determine load endurance limits for historic masonry arch bridges;
- observe the failure behaviour of the arch during cyclic testing.

The arches will be tested in cyclic experimental for:

i. Cyclic loading progressively increasing - 0.5 kN increments every 200 cycles - over a number of loading intervals (intervals are either 200, 400, 800 or 2000 cycles) up to failure.

ii. Cyclic loading at the 25% of failure load -2.5 kN±1.0 kN - for 80,000 cycles and then near failure load - 10 kN±2kN - until failure by fatigue (800 cycles).

iii. Cyclic loading at the 50% of the failure – 5 kN±1 kN - for 50,000 cycles and then at 75% - 7.5 kN±1.5 kN - until failure -3,800 cycles-, unless the arch had already failed or was deemed to have permanently deformed and become unusable.

The collected data will be significant from a collapse mechanism, ultimate displacement and seismic performance of masonry arches point of view.

For dynamic experimentation the test procedure involves the application of a cyclic load, which is normally used to assess the dynamic behaviour of structures. Furthermore, all dynamic tests will be conducted at a frequency of 1 Hz to simulate a vehicle travelling at 30 MPH (≈50 km/h), but this frequency is also representative of the frequency content of European earthquakes.

5.2.3.3 Monotonic test, setup description

Arches are made of a single ring of Victorian bricks of dimensions 225 x 115 x 75 m with three rows of holes. The arches are built by the use of a plywood formwork, laying bricks on the narrow side so that each arch is 115 mm thick and 225 mm wide.

The span measures 2000 mm and the arch will be 500 mm high above its lowest point.

A pair of concrete abutments is used to support the arches with 45° springing resting on bridge bearings laying on a 300 mm wide steel C-section. While one of the abutments will be fixed to the C-section by other smaller channel sections welded together, the other will be left free so as to undergo controlled movement and simulate the relative motion of the abutments. The movement will be controlled by a hydraulic jack that, at the beginning of the test, prevents any movement and provides the necessary restraint, as friction between the concrete block and the base will be not sufficient on its own. Reliving the pressure in the jack, the abutment will be able to slide away from the fixed end at a constant pace of 5 mm per step, these being read by an LVDT placed behind the abutment.

The arch fill will be simulated by sand bags with a total weight of 150 kg.
Seven LVDTs will be positioned along the arch, perpendicularly to the curvature, while two 10 t load cells will be positioned at the abutment to measure the horizontal thrust induced by the arch action.

Low density SRPs with high carbon steel cord with micro-fine brass coating – 3X2 Hardwire – will be used for the reinforcement of Arch 2 and Arch 3. The cord is made of 3 straight filaments wrapped by 2 filaments at a high twisted angle.

A single layer of SRP sheet sized 225 x 223 mm will be applied to either the intrados (Arch 2) and the extrados (Arch 3) by bonding with a thin layer of basecoat.

In reinforced arches (Arch 2 and Arch 3), strain gauges will be used to measure the strain of SRP in the same locations where LVDTs will be positioned (Figure 5.10). Strain gauges will be applied directly onto the arch after removing a small section of the SRP sheets.

The test procedure consists in loading the arch with the sand bags simulating the fill and then proceeding to move one abutment away from the other, monitoring the formation of cracks. Three cracks form in the first place creating a stable three-hinge system; the formation of a forth crack signals the failure of the arch: the test is stopped and the sand bags removed to assess the recovery capacity of the arch.

![Figure 5.10 - Position of strain gauges in reinforced arches, (Ng 2008).](image)

**5.2.3.4 Cyclic test, setup description**

The three two-ring arches are built using handmade reclaimed bricks from around 1870 with average dimensions 226 x 108 x 69 mm laid on the narrow side by me using NHL 5.0 in a 1:3 (cement:aggregate) mix where the aggregate size was < 5 mm and the average joint thickness was 10 mm. To allow as much of an overlap of the bricks in one ring relative to the other, the end brick at opposite ends of each ring was split along its major axis. The clear span of the arch is 2000 mm and the rise at the crown is 500 mm. Abutments are the same as described in section 5.2.3.3 and are connected by 25 mm diameter bars to prevent relative movements, while the bridge fill is simulated by sand bag resting on the arches.

Arch 1 (Figure 5.11) will be loaded using a hand operated hydraulic jack to bring the static loading up to 5 kN. Cyclic load increasing at 0.5 kN/min up to 5 kN will be applied by a computer-controlled hydraulic ram. For each load interval, after completing a certain number of cycles, the test will be paused to record the formation of cracks. The number of cycles in each interval will be progressively increased up to 2,000 cycles.

The test will be repeated increasing both static and cyclic load to 10 kN. Linear Variable Displacement Transducers (LDVTs) connected to the computer scanners will be used to measure the relative displacement of bricks at critical locations.
Arch 2 (Figure 5.12) will be loaded following the same procedure as Arch 1 except for the static load cell, which will be removed. The cyclic load of the ram will be reduced to 2.5 kN ± 1.0 k. After 80,000 cycles, the load will be increased to 10 kN ± 2.0 kN and the arch tested for a further 800 cycles until failure.

Arch 3 (Figure 5.13) will be tested following the same procedure, with the cyclic load equal to 5 kN ± 1.0 kN; two of the LVDTs at each of the quarter points will be re-orientated to measure more accurately the opening up of the radial cracks, (this will be done at the 20,000 cycle mark). After 50,000 cycles, the load will be increased to 7.5 kN ± 1.5 kN.
5.2.3.5 Materials

**Bricks**: recycled Victorian bricks are used.

**Mortar**: a hydraulic lime mortar NHL 3.5, mix sand:lime:water 3:1:1, is used. Compressive tests are performed on two batches samples, one made of the same mortar as Arch 2, the other made of the same mortar as Arch 3.

**Masonry**: the masonry is characterised by compression tests on stacks of 5 bricks. Wrench tests are performed to determine the bond strength of the masonry.

**Mortar resin**: maximum compressive load equal to 10.18 kN.

**SRP**: the characterisation of SRPs is performed by Castori (2006) by tensile and pull-off test. For 3X2 cords, the mean value of the tensile strength is 2228 MPa, with ultimate strain 25948 με. The Young modulus of the cord is 153,565 MPa, namely 26% lower than that of the wires due to the twisting in the cord. Tensile bond strength is 1.57 MPa with failure in the substratum.
5.3 ROOFS

5.3.1 Partner n° 2 and 17 - BAM and ZRS

5.3.1.1 Objectives

Historical earthen structures usually exhibit one to two storey levels (Figure 5.14), with either pitched, flat roofs or vaults. Excluding non-seismic areas, flat roofs are most common in the Southern-Mediterranean (Morocco, Algeria), whereas pitched roofs are found in Northern mediterranean areas (Abruzzo, Marche, Italy) and colonial Spanish architecture in the Americas.

![Typical historical adobe buildings in earthquake zones: left one storey with a heavy ceramic tile roof, Central Chile and right two storeys with a lightweight material roof, California, U.S.A.](image)

Pitched roofs in adobe or other earth construction are comparable to those in other types of masonry constructions. Flat roofs are usually built of wooden primary beams (often palm, cedar or poor quality poplar) resting on either a ring beam or directly on the walls. A system of secondary beams or smaller elements rests on the primary beams. The cover consists of a layer of compacted earth, of 10 to 20 cm thickness. Since roof maintenance is often carried out by simply adding earth onto the roof structure instead of preventing leakage causes and improving water runoff, complete roof collapses are frequent when earthquakes occur.

While a flexible and lightweight roof is often claimed to be an appropriate solution for adobe structures, (Lopez et al., 2002; Blondet et al., 2003), the presence of a dead load on the walls might provide additional stability against horizontal loads, as long as the load of heavy elements resting on the walls is well tied to them. Historical earthen houses with pitched roof were often originally built with ceramic roof tiles. In a number of refurbishment or repair campaigns, ceramic roof tiles were removed and replaced with lightweight roofing materials such as corrugated metal sheets or tar paper panels or sheets, thus considerably reducing the dead load on the walls.

The roof design of historical earth buildings has to be seen in the context of a diaphragm system for the load distribution of the roof’s dead load on the walls. As it was pointed out by Tolles et al. (2002), thick adobe masonry walls and other thick earth walls have a different out-of-plane and in-plane behaviour as conventional modern masonry buildings. Thick earth walls (span/thickness ratio < 5) often have substantial stability in the out-of-plane direction and are more sensible in the in-plane direction. Accordingly, an existing diaphragm system in a historic earth building has to take on in-and out-of-plane loads. Another aspect concerns the stiffness of a diaphragm. If a diaphragm is stiff loads will be transferred from the out-of plain walls to the in-plane walls. Due to the fact that the in-plane performance of thick walled historic earth buildings is low, overloading due to diaphragm load redistribution and subsequent damage of in-plane walls can occur in case of stiff diaphragms, (Tolles et al., 2002).

Traditional construction methods in earthquake zones was often already realized with wooden horizontal bond beams on top of the walls which can function as a diaphragm system together with the roof construction if the roof is appropriately fixed to the bond beam and the bond beam is
There is a need for new knowledge-based approaches to protect cultural heritage from earthquake-induced risk. This involves understanding how structural design can effectively improve the performance of earthen buildings during seismic events.

**Figure 5.15 - Horizontal bond beams in adobe masonry (arrows).**

The goal of the BAM and ZRS contribution to work package 5.1 is to evaluate the structural performance of a simple model structure with a partially stiff floor (partial diaphragm) and with a stiff floor (full diaphragm) in conjunction with a bond beam and a light or heavy dead load representing a lightweight or heavyweight roof. The results are expected to show the influence of the type of roof construction (light or heavy weight) in context with a horizontal diaphragm on the behaviour of earth buildings in earthquakes.

### 5.3.1.2 Experimental programme

The study will compare the performance of three model structures without openings. The model structures represent one storey buildings and will consist of a four walled adobe masonry with a size of 1.5 x 1.5 m² base area, a wall height of 1.2 m and wall thickness of 25.0 cm.

Three types of model structures will be tested:
- partial diaphragm, lightweight roof;
- partial diaphragm, heavyweight roof;
- full diaphragm, lightweight roof;
- full diaphragm, heavyweight roof.

The wooden bond beam will be fixed to the masonry by vertical pins, which will be grouted into the masonry. The bond beam will be tied together in the corners. The dead load will be adjusted to represent a lightweight roof (e.g. corrugated metal sheet) and a ceramic tile roof, respectively.

The four structures will be exposed to unidirectional dynamic loading inducing shear forces in the in-plane direction and flexural loads in the out-of-plane direction. Additionally, the displacement and load will be monitored during the experiments.

This experimental test will be connected to WP6 (connections and dissipative systems with early warning).

### 5.3.2 Partner n° 4 - NTUA

#### 5.3.2.1 Experimental programme

The experimental campaign will consist on testing a subassemblage and full buildings on an earthquake simulator. This tests will allow to check methods of improving the behaviour (and, hence, the effect) of "horizontal", elements to the seismic behaviour of buildings.
The subassemblage consists of two parallel walls (three leaf stone masonry) connected with a timber floor. The subassemblage will be tested twice, parallel to the longitudinal axis of the walls. Test No A: As built. Test No B: Grouting of walls and strengthening of the diaphragm action of the timber floor (by placing cross planks).

Furthermore, the tests on entire buildings, we also allow to check the effect of improving the diaphragm action of timber floors, as this is going to be one of the intervention techniques to be investigated.

This experimental test will be connected to WP7 (systemic improvement of overall seismic response).

5.3.3 Partner n° 6 - UMINHO

5.3.3.1 Objectives
Timber roof trusses will be tested under vertical, symmetric and asymmetric loads. The influence of the number and location of point loads, without and with eccentricity relatively to joints, will be assessed. Afterwards, the trusses will be strengthened using techniques as the ones regarding the use of wood based products and steel elements. Strengthening of timber trusses normally involves the rafter and tie beam connections. The intervention will be evaluated in terms of safety (strength and stiffness), lifetime performance, easiness of implementation and susceptibility to execution errors.

5.3.3.2 Task description
This task aims at improving the understanding of the behaviour of existing timber roof structures and to support eventual retrofitting interventions. The study includes a preliminary structural survey to assess geometries, materials and on site failure modes, as well as the experimental testing of full-scale samples of trusses retrieved from old constructions and joints. Aiming to assess the overall behaviour of traditional timber roofs, three-dimensional full-scale timber roof structures will be evaluated taking into account, not only the system effect provided by the covering substructure, but also the interaction with supports.

A preliminary survey will be performed on different roof trusses to collect data on the geometry, connections solutions, state of conservation, wood species and load parameters. In practice, this survey follows the activity of the team project in their everyday actions since some years ago. Based on this survey, some representative examples will be retrieved and transported to the laboratory. It is important to point out that some examples have been already stored in the laboratory, namely, three Queen-post timber trusses collected in 2008. Before the load-carrying tests, a detailed non-destructive evaluation of the elements alongside with visual strength grading will be performed to assess the mechanical and physical characteristics of the timber components of the trusses.

The selected trusses previously will be subjected to cyclic tests, under symmetric and asymmetric loading for different load levels. The influence of the number and location of point loads, without and with eccentricity relatively to joints, will be also assessed. The connections between the trusses and the supporting walls are an important issue that will be analysed under the project. In particular, during the load-carrying tests on the full-scale trusses, different support arrangements will be considered to simulate different types of connections between the trusses and the walls through strength and stiffness variation. Also the influence of wood degradation near the supports will be assessed. Afterwards, the trusses will be strengthened by using only traditional strengthening techniques, as the ones based on the use of wood based products and steel elements. The use of glues or composite materials will not be a choice. The efficiency of the different strengthening techniques will be assessed comparing the performance of the strengthened trusses with the original ones.
5.3.3.3 Expected results

This Task will allow acquiring knowledge on the design and assessment conditions that lead to an inadequate performance of traditional timber roofs, and on the efficiency of retrofitting interventions using traditional materials. Thus, a significant improvement on the design, execution and maintenance of timber roof structures is also expected.

5.3.4 Partner n° 10- ENA

5.3.4.1 Introduction

Traditional construction is based primarily on the use of a structure including load-bearing walls made out of stone, earth bricks/adobe, pisé and floors in wooden beams out of cedar and resinous white-cedar (thuya) and palm wood, Figure 5.16, in some regions (east and south of Morocco).

![Figure 5.16 - The palm wood, Cutting and preparation of palm beams, Example of realisation in Figuig (east of Morocco).](image)

In general, the floors, Figure 5.17, and roofs are constructed of timber joists in the round overlaid with solid timber boarding. The boarding is covered with a layer of mud, sand, and lime and finally sealed with an impermeable, lime-based render called “Dess” in Salé medina for example.

![Figure 5.17 - Example of floor construction in Fez medina.](image)

The sealing of the roofs in Fez medina, are constructed according to the following procedure:

- 5.0 cm layer of mortar, not fermented, containing sand and lime (sand ½ and lime ½);
- the mixture is spread out over a layer of crusher run aggregate or infill, then rammed with water until water remains on the surface and creates an impermeable film;
- solid bricks (‘Dfira’) spread out as a layer, Figure 5.18.

Figure 5.18 - (a) Example of ‘Dfira’ spread out as a layer in Fez. (b) Example of Tataoui style of roof (south of Morocco).

### 5.3.4.2 Experimental programme

**Wood Characterization: Experimental tests on wood**

First, experimental tests will be carried out on samples of cedar taken from a forest nearby the medina of Rabat-Salé to estimate the density, the components for Young’s modulus: radial, transversal and longitudinal; the components for the Poisson ratio; and the components for the shear modulus.

Secondly, experimental tests will be performed on various samples of hardwood cedar and resinous white-cedar wood taken from buildings within the Médinas of Fez, Rabat and Salé.
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