Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings.

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Spain

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Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings.
DECLARATION

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Thank you to Elitsa Ivanova Teneva for having helped me in the laboratory activities, and RosaMaria Codispoti for having encouraged me to undertake the SACH master

A special thanks to my family.
ABSTRACT

Rammed building is an ancient building technique spread in all the five continents as well as other earthen construction technique. Currently almost the 30%-40% of the world population live in earthen dwellings. The rammed earth diffusion is due to the large availability of the row material that doesn't require any processing, hence a low skilled manpower is required. Nowadays this technique is appreciated from the point of view of environmental and social sustainability, because of the absence of energy consumption, none of the less for its affordability.

Rammed earth is characterized by a satisfactory compressive strength, whereas the tensile strength id very low. The latter factor often is the cause of severe and damage collapse due to earthquake since the diffusion of earthen construction mostly, involves seismic prone area.

The aim of this work is to evaluate the effectiveness of the glass fiber mesh as seismic reinforcement and strengthening system in existing and new Rammed Earth building as well. For this purpose, compressive test and shear test are performed on three reduced scaled models namely: an unreinforced sample, a retrofitted sample by mean of glass fiber mesh applied on the two surfaces of the wall, a reinforced sample by two layer of glass fiber mesh, internally placed in the thickness of the scaled wall model.

From the present experimental work aroused that glass fiber mesh allows to increase considerably the ductility of rammed earth wall, while the strength is conferred by the compacted soil. Furthermore, the internal reinforcement demonstrates also a greatly contribution to the performance of rammed earth constructions in compression and in tension as well, even though further experimental studies are required. In conclusion glass fiber mesh can be considered an effective and promising strengthening option for historical rammed earth constructions.
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings
RESUMO

El tapial es una antigua técnica constructiva que, como otras técnicas de construcción en tierra, está extendida por los cinco continentes.

Actualmente casi el 30%-40% de la población mundial habita en viviendas de tierra.

El tapial debe su gran propagación al uso de un material que no requiere de procesado, y por tanto, necesita poca mano de obra.

Hoy en día, esta técnica desde el punto de vista medioambiental y sostenible, valorando la ausencia de consumo de energía y a parte de su asequibilidad.

El tapial se caracteriza por tener una satisfactoria resistencia a compresión, pero una resistencia a tracción muy baja. Este último factor causa a menudo daños severos y colapsos producidos por terremotos, ya que la construcción en tierra está mayormente distribuida en lugares altamente sísmicos.

El objetivo de este trabajo es evaluar la eficacia del uso de una malla de fibra de vidrio como refuerzo sísmico en edificios de tapial nuevos y existentes.

Con este fin, se realizan experimentos a compresión y a cizalladura en tres modelos de escala reducida: en una muestra sin refuerzo, en una muestra con refuerzo de fibra de vidrio aplicada en las dos superficies de la pared y en una muestra con dos capas de malla de fibra de vidrio situadas dentro del grosor del modelo a escala.

Del presente trabajo experimental se deduce que la malla de fibra de vidrio permite un crecimiento considerable de la ductilidad del muro de tapial, mientras la resistencia se confiere a la tierra compactada.

Asimismo, el refuerzo interno también demuestra una gran contribución al rendimiento de las construcciones de tapial en compresión y en tensión, aunque más trabajos experimentales deben ser realizados. En conclusión, la malla de fibra de vidrio puede considerarse una técnica de refuerzo efectiva y una prometedora opción para los edificios históricos.
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings
RESUMÉ

STUDIO SPERIMENTALE SULL’INFLUENZA DEI RINFORZI TESSILI SUL COMPORTAMENTO STRUTTURALE DI MODELLI IN SCALA RIDOTTA DI EDIFICI IN TERRA BATTUTA

La terra battuta è un’antica tecnica costruttiva presente in tutti e cinque i continenti, così come altre tecniche e costrutti in terra cruda. Attualmente circa il 30%-40% della popolazione mondiale abita in dimore in terra battuta. La diffusione della terra battuta è dovuta alla grande disponibilità delle materie prime che non richiede, peraltro, un complesso processo di lavorazione, dunque non è richiesta nemmeno una manodopera che sia particolarmente qualificata. Attualmente questa tecnica è molto apprezzata dal punto di vista della sostenibilità ambientale ed economica, per l’assenza di consumo energetico, cionondimeno per la sua economicità. La terra battuta è caratterizzata da una buona resistenza a compressione, ma da una bassa resistenza a trazione. Quest’ultima è spesso la causa di grossi danni e collassi in caso di sisma, poiché la diffusione di questa tecnica costruttiva spesso coincide non zone ad rischio sismico. Lo scopo del presente lavoro è di valutare l’efficacia di maglie in fibra di vetro come sistema di rinforzo antisismico in costruzioni di terra battuta esistenti, così come in quelle di nuova produzione. A tal fine prove di resistenza a taglio e compressione sono state effettuate su tre modelli di muro in scala ridotta: uno non rinforzato di riferimento, un provino rinforzato da due maglie in fibra di vetro applicate sulle due superfici del muretto, un provino rinforzato con due strati posti all’interno dello spessore del provino stesso. Dal presente lavoro sperimentale è emerso che le maglie in fibra di vetro permettono di aumentare notevolmente la duttilità, mentre la resistenza è affidata alla terra compatta. Inoltre il rinforzo interno contribuisce notevolmente al buon comportamento delle costruzioni in terra battuta in compressione e tensione, nonostante siano necessari ulteriori studi sperimentali. In conclusione le maglie in fibra di vetro possono essere considerate una valida alternativa come sistema di rinforzo per costruzioni storiche in terra battuta.
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1. INTRODUCTION

1.1. Motivation
Rammed building is a very ancient building technique spread in all the five continents as well as other earthen construction technique.
Currently almost the 30%-40% of the world population live in earthen dwellings.
The rammed earth diffusion is due to the large availability of the raw material that doesn’t require any processing, hence a low skilled manpower is required.
Nowadays this technique is appreciated from the point of view of environmental and social sustainability, because of the absence of energy consumption, none of the less for its affordability.

1.2. Objectives
The aim of this work is to evaluate the effectiveness of the glass fiber mesh as reinforcement and strengthening system in existing and new Rammed Earth building as well.
For this purpose, compressive test and shear test are performed on three reduced scaled models namely: an unreinforced sample, a retrofitted sample by mean of glass fiber mesh applied on the two surfaces of the wall, a reinforced sample by two layer of glass fiber mesh, internally placed in the thickness of the scaled wall model.

1.3. Structure of the work
Firstly, the most recent and remarkable experimental researches about rammed earth properties and reinforcement methods are presented.
Subsequently, the experimental methodology a procedures carried out in this work are described, including production process and test assembling.
Further on, the obtained experimental results are presented and discussed.
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings
2. STATE OF THE ART

2.1. Rammed earth constructions

The rammed earth building technique consist in rising monolithic wall by adding and compacting layer of wet soil between removable wooden boards.

It is historically spread worldwide, the names, used in different languages, to identifies this technique, refers to the compacting action of ramming: ‘pisé’ in French, ‘tapial’ in Spanish, ‘taipa’ in Portuguese, ‘terra battuta’ in Italian and ‘Stampflehm’ in German.

The timber formwork has the function to stand the load due to the compaction effort, hence it needs to be provided with some stabilizing devices.

The execution of the walls occurs by recourses, thus the traditional timber formworks, that surrounds horizontally the walls, are moved vertically to the next level once that the current layer of soil has been compacted. The crawling formwork are usually supported by the built portion of wall below, in this case putlogs holes are visible on the surfaces of the constructions. Seldom the formwork are self supporting and provided of scaffoldings. The compaction used to be traditionally applied manually by mean of wooden elements. In modern constructions, continuous formworks and pneumatic or vibratory rammers are utilized (Figure 1).

![Figure 1](image1.png)

Figure 1 c)Traditional and a) modern rammed earth building technique; b) crawling formwork

The origins of earthen construction techniques have been found in China (221-581 A.D.) and in the Mediterranean areas, particularly in the Iberian peninsula and northern Africa by the Phoenicians (800 B.C.).

Later, Islamic military fluxes again built earthen fortresses in the Iberian Peninsula.

Spanish and Portuguese settlement brought earthen techniques building in Southern America.

European settlers spread the technique in Australia and New Zealand and north America.

It has to be noticed that most of the earthen building diffusion coincides with seismic prone areas. (Figure 2)
2.2. Experimental research context

In this paragraph, some remarkable researches about the topic in exam are presented. It has to be noticed that the most significant campaigns about shear tests and mechanical characterization on reinforced rammed earth construction are quite few and recent. The researches studies are presented according to the typology of test program adopted, that are the same tests typology performed in the present work. Nonetheless, the reinforcement technique proposed are noteworthy as well as the data collected about the mechanical characterization of the rammed earth.

2.2.1. Mechanical properties

As strength criterion for rammed earth Mohr Coulomb is adopted as commonly happens for soils. It states that if at any point on any plane within the soil the shear stress becomes equal to the shear strength of the soil then failure will occur at that point (Craig, 2004).

\[ \tau = \sigma \tan \varphi + c \]  

(1)

The latter equation represents the Mohr Coulomb envelope, intended as failure envelope in normal-shear space. Shear stress is a linear function of the normal stress, the envelope is tangent to the Mohr coulomb circles that represent the stress state at shear failure on a failure surface (fig.3).

In the equation n.1 \( \tau \) is the shear stress, \( \sigma \) is the normal stress, \( \varphi \) and \( c \) are material properties or shear strength parameters, specifically \( \varphi \) is the stress dependent component defined as \textit{angle of shearing resistance} or \textit{angle of internal friction} since it depend from the frictional resistance in sliding between the particles, \( c \) is called \textit{apparent cohesion} and it's the resultant of chemical, electrochemical and suction forces within the soil.
The strength parameters of the soil mixture implied in this work are reported in Table 1 Mohr Coulomb properties. These parameters has been determined during a previous research carried out by the UPC in the campus of Terrassa.

<table>
<thead>
<tr>
<th>Mohr Coulomb properties</th>
<th>c [Mpa]</th>
<th>Φ[°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.64</td>
<td>38</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 Mohr Coulomb properties

Rammed earth layered structure is not homogeneous, it shows increasing density up along the same layer and different values between the layers, specifically, the lasts layer are weaker because less confined than the ones underneath. Moreover, the interfaces between the layers has to be considered. From an experimental research carried out by Q.-B. Bui, J.-C. Morel (2009), emerged that as long as the layer remain adherent one to each other, rammed earth can be considered as an isotropic material.

Unconfined compressive tests, unloading-reloading cyclical tests were carried out on Equivalent Volume of Rammed earth and on homogenized Compressed Earth Blocks, along both directions perpendicular and parallel to the layer.

The failure modulus, defined as the ratio between the maximum stress and the corresponding deformation, resulted to be equal one to each other, at roughly 70 MPa, in the two directions.

The difference between the compressive strengths obtained in the two directions, was less than 10%.

The average elastic modulus for unloading and reloading were quite the same in the two directions for preloading load of 0.4 MPa.

Further investigation are needed on site buildings, regarding the anisotropy of rammed earth, however in the present work, as it’s shown hereafter, the scaled wall specimens can be considered isotropic since they are constituted only by one compressed soil layer. Noteworthy is the work published in 2014 by Miccoli et al. In latter, an experimental campaign aimed to compare the mechanical properties achieved in the main earthen building techniques is presented. Moreover an exhaustive literary review is proposed. Here after the attention is focused on the information concerning the rammed earth.

<table>
<thead>
<tr>
<th>References</th>
<th>Bulk density (kg/m3)</th>
<th>Compressive strength (MPa)</th>
<th>Young’s modulus(Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dierks K. 2000</td>
<td>2100–2300</td>
<td>2.40–3.00</td>
<td>650</td>
</tr>
<tr>
<td>Bui Q.B. 2009</td>
<td>1800</td>
<td>1.00</td>
<td>90–105</td>
</tr>
<tr>
<td>Rölen 2010</td>
<td>1700–2400</td>
<td>1.50–4.00</td>
<td>750</td>
</tr>
<tr>
<td>Hall M. 2014</td>
<td>2020–2160</td>
<td>0.75–1.46a</td>
<td>nd</td>
</tr>
<tr>
<td>Lilley DM. 1995</td>
<td>1870–2170</td>
<td>1–80–2.00</td>
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<tr>
<td>Jaquin PA 2007</td>
<td>nd</td>
<td>0.60–0.70</td>
<td>60</td>
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<tr>
<td>Maniatidis 2007</td>
<td>1850</td>
<td>3.88</td>
<td>205</td>
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<tr>
<td>Ziegert 2003</td>
<td>1850</td>
<td>2.46</td>
<td>160</td>
</tr>
<tr>
<td>Ziegert 2003</td>
<td>1763–2027</td>
<td>0.62–0.97</td>
<td>60–70</td>
</tr>
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</table>

Table 2 Literary review about rammed earth's mechanical properties (Miccoli et al. 2014)
The soil implied to produce the sample of the experimental campaign was characterized by a shrinkage of 0.5%, a bulk density of 2190 kg/m$^3$ a particle size range 0-16mm. The rammed earth specimen were produced with a moisture content of 10% in 10 cm deep soil layer manually compacted.

Compression test was carried out under force control by mean of a 1 MN universal testing machine on wallets of size of size 500 x 500 x 110 mm$^3$. LVDTs sensors were utilized to measure the displacements. The obtained results are namely: Compressive strength equal to 3.73 MPa, Young modulus of 4143 MPa, Poisson modulus equal to 0.27.

2.2.2. In-plane cyclic shear-compression

In 2016 the BAM institute in collaboration with the Institute of Theoretical and Applied Mechanics (ITAM) of Prague, inside the framework of a NIKER project, carried out an experimental campaign about the shear behavior of Rammed earth walls (Miccoli 2016).

The experimental research was followed by a work of numerical modeling by mean of a finite element method software.

The experimental program foresaw to perform in plane compression shear tests on three full scale rammed earth wall samples. The size of the walls was 1300mm x 1050mm x 250mm, the bulk density equal to 2190 kg/m$^3$, maximum particle size of 16 mm. The soil mixture were produced with a moisture content of 9-10% by mass, than poured in a plywood formwork and compacted in thirteen layer. The compaction process were performed using a mechanical rammer, that lowered each layer from an initial height level of 150 mm to 100 mm. The moisture content at the testing moment was about 2-3% by mass.

Preliminary static tests were carried out on wallets with a size on 150mm x 150mm x 100mm to determine the following mechanical characteristics:

- Uniaxial compressive strength $f_c = 3.73$ N/mm$^2$;
- Shear strength $f_{max} = 0.70$ N/mm$^2$;
- Young modulus $E = 4143$ N/mm$^2$;
- Shear modulus $G = 1582$ N/mm$^2$.

Figure 4 Specimen production phases, Miccoli 2004
For the in plane shear compression test, the test-set up (Figure 5 a) was designed in order to allow the walls to behave like a cantilever beam, clamped at the base and loaded at the top. Specifically, at the top of the walls a C profile steel beam was placed in order to transmit the loads from the actuator to the samples and, at the same time, to avoid the sliding. A constant compressive tension $\sigma$ of 0.56 N/mm$^2$, equal to the 25% of the compressive strength, was imposed by mean of three hydraulic actuators connected to the capping beam. The shear test was carried out imposing increase of 2.5 mm each step in the displacement amplitude with a frequency of 0.1 Hz. The peaks in the displacement amplitude were repeated three time each step.

Displacements, flexural and shear deformations were measured by six linear variable differential transformers LDVT sensors, while the loads were registered by load cells linked to the actuators.

The test was stopped when the measured load started to decrease.

During the test execution four limit states were identified and the corresponding lateral load $H$, lateral displacement $\delta$ and rotation angle $\theta$ were registered.

Specifically, the first non linearity, was qualitatively observed, the second one, coincided with the opening of the first crack, while the third limit state corresponded to the formation of a diagonal strut associated to the appearance of crossing cracks throughout the rammed earth layers, nonetheless the maximum values of lateral load an displacement are reached.

The fourth limit state corresponded to the collapse of the walls.

The failure mode Figure 5 b) was constituted by the typical diagonal shear cracks. In two of the three sample, X shape cracks appeared, while a diagonal crack marked the third sample.
A FEM numerical model, based on the experimental results of the described campaign, was elaborated (Figure 6). Total strain rotating crack model for the rammed earth layers and Mohr Coulomb failure criterion for the layer interfaces, were adopted as constitutive laws.

In the same year, 2016, M. E. Arslan et al, from the university of Düzce in Turkey, published their work about the assessment of the structural behavior of rammed earth walls under lateral cyclic loading in comparison with stabilized rammed earth walls and other traditional masonry building techniques. Specifically the experimental program was carried out upon six sample with dimensions of 150cm x 150 cm x 20 cm: one made of non stabilized rammed earth, three made of stabilized rammed earth, one of brick masonry and one of aerated concrete blocks (Table 3 Specimens identification (Arslan 2016)).
The compressive strength of each mixture was determined by mean of compression test executed on three cylindrical sample with dimensions of $\Omega150 \times 300$ mm. From the latter tests emerged that the stabilizing additives increase the compressive strength, moreover glass fiber allow to decrease the amount of cement in view of an acceptable loss of resistance.

The lateral reversed cyclic loading set-up was constituted by a steel frame hinged at the joints that was surrounding the walls. A servo-hydraulic actuator transmitted the lateral loads to the sample. The presence of the hinges allowed to measure the real values of load and displacement of the wall inside the frame without the contribution of columns and beams. The reversed displacement cycles were applied with the following drift ratios each three cycles 0.15%, 0.20%, 0.25%, 0.35%, 0.50%, 0.75%, 1.00%, 1.40%, 1.75%, 2.20%, 2.75%, 3.50%, 4.00%, 4.50%, 5.00%, 5.50%, 6.00%, 6.50%.

The lateral loads was measured by mean of a load cell with 500 kN of capacity, moreover a linear potentiometric displacement transducer (LPDT) was applied.

The comparison between the load displacement envelope curves shows that the higher lateral load carrying capacity is obtained by the Masonry Aerated Concrete wall, followed by the rammed earth wall stabilized with cement and glass fiber, whereas, the lowest load carrying capacity was obtained in masonry hollow bricks wall.

Similarly, higher values of total energy dissipated were associated to the aerated concrete wall and the rammed earth wall stabilized with cement and glass fiber.
Furthermore the secant stiffness, determined as the slope of the line that connect the peak load in push and pull direction for each cycle were determined in correspondence of each drift ratio. In fig. is represented the stiffness degradation, the secant stiffness decrease for increasing drift ratio. The highest initial stiffness is obtained for the two cement stabilized rammed earth walls.

Regarding the failure mode, for the two 10% cement stabilized walls, the typical crack pattern of shear walls laterally loaded, constituted by X-shaped cracks was observed, while the non reinforced rammed earth wall and in the cement and blast furnace slag stabilized wall, sliding plane coinciding with the separation rammed earth layer where more evident.

In the hollow brick masonry walls shear surfaces in the joints appeared after that diagonal cracks appeared. In the aerated cement wall, scattered diagonal cracks characterized the failure mode.

### 2.2.3. Monotonic lateral loading

Subsequently to the earthquake occurred in the Yunnan Province in China in 2014, the Normal University of Beijin and the Academy of Disaster Reduction and Emergency Management of Beijin, undertook an experimental campaign in order to refine a strengthening technique that could enhance the seismic performance of shear rammed earth walls (Liu et al. 2015).

Lateral load capacity of four specimens (2400 mm x 2100 mm x 600mm) and ductility were evaluated by mean of monotonic later load test. One of the specimen was left unreinforced as reference, the other three were reinforced with 20 cm wide tarpaulin strips. The latter were bonded on two walls in horizontal direction, respectively, in one and two overlaid layers, while, on the last specimen they were placed in diagonal directions.

Moreover, preliminary debonding test were carried out to evaluate the performance in term of strength, workability, ductility, availability and cost of different fiber materials and adhesives, namely: canvas, bamboo shavings, and tarpaulin as fiber and: epoxy adhesive, sodium silicate and NF compound as bonding materials.
Finally Tarpaulin and NF compound were chosen as reinforcing materials. The rammed earth walls were constructed by local builders with the soil commonly used in the region of Yunnan for the construction of rural dwelling.

The test set-up was arranged as follows: the walls were connected to a concrete footing provided of coarse stone to reproduce the traditional technique used in order to better connect the walls to the foundations.

![Reinforced specimens and test set-up (Lui 2015)](image)

On top of the specimens, a concrete beam was casted in situ in order to transfer lateral and vertical loads, specifically a constant compressive stress of 0.06 MPa were transmitted by mean of a steel beam placed over the top concrete beam to simulate the weight of the roof. The inertial seismic action was produced as lateral load at the end of the concrete loading beam.

The test was executed in load control with increments of 2kN from the lateral load hydraulic jack. The load were measured by load cells connected to the jacks, lateral displacements were registered by displacements gauges, and the strain of the tarpaulin was measured by strain gauges.

The unreinforced wall reached a maximum load corresponding to an ultimate displacement of 57 mm before showing a softening behavior. In this case the crack pattern is characterized by the appearance of diagonal cracks starting from the loading point and propagating until the failure, clearly occurred due to shear.

The wall reinforced with two layer of tarpaulin horizontally placed, achieved an increase of 22% of lateral load carrying capacity, and 63% of ultimate displacement in comparison with the unreinforced specimen. Beneath the strips of reinforcement, the wall presented an x-shaped crack pattern.

A similar but slighter failure mode was showed by the wall strengthened with three layer of tarpaulin, that besides, presented a 75% higher load capacity and a 38% bigger ultimate displacement.

Finally the wall reinforced with diagonally placed tarpaulin strips, exhibited scattered diagonal crack along the strips starting from the loaded corner to the diagonally opposite one, with increases in load carrying capacity and ultimate displacement respectively of 82% and 29%.

The described reinforcements provided enhance in the stiffness and in the strength of the walls that, thanks to the bonding effect of the strips, could maintain its integrity and reach the maximum load capacity. In spite of the presence of crack, and subsequently to the loss of bonding, underwent to higher deformation.
2.2.4. Diagonal compression test

Rui A. Silva, Daniel V. Oliveira, Lorenzo Miccoli and Luc Schueremans, published a work about modeling rammed earth walls under shear loading by mean of FEM method.

The experimental program foresaw to test in axial and diagonal compression 5 rammed earth wallets of 499mm x 505 mm x 117 mm, respectively.

The soil mixture used in the BAM institute of Berlin to executed samples, had a bulk density of 2190 kg/m³, moreover the particle composition was about 11% clay, 25% silt and 64% sand and gravel, respectively.

The axial compressive test was performed under displacement control, imposing that the failure would have occurred in a range of 20-30min. LVDTs were applied on the surfaces of the sample in order to measure vertical and horizontal deformation. The following average values of mechanical characteristics were obtain from the described test:

- Compressive strength ($f_c=3,7$ N/mm²);
- Young modulus $E_0= 4207$ N/mm²;
- Poisson ratio $\nu=0,27$

The diagonal compression test were carried out in accordance with the standard ASTM E 519-10. LVDTs sensors were placed on both surfaces of the wallettes (Figure 11). The average values obtained were: shear strength $f_s=0.70$ N/mm²;

- shear modulus $G_0=1582$ N/mm².

The curves stress strain for the shear in Figure 11 b, show that that an early peak of shear stress appears at the end of the elastic phase of the  is followed by a long hardening branch.

![Figure 11 Diagonal test (Miccoli 2015): a) test set-up; b) Shear stress-shear strain curves](image)

The crack pattern exhibited initial cracks in correspondence of the early peaks of stress shear, subsequently, diagonal cracks developed in the middle of the specimen from the bottom to the top corner. Cracks appeared also between the rammed layers’ interfaces.
3. Experimental program
In the present chapter the properties of the materials implied and all the operations involved in the production of the samples are illustrated.

3.1. Material Characteristics
The behavior of the soil as building material is not deeply defined and investigated as much as other building materials, most of the knowledge comes from the geotechnical engineering. References from literature, i.e: handbooks, guidelines, technical documents, often provide incomplete, and contradictory indications. More reliable, results come from experimental researches, even though criteria that could universally be applied in a standardized way are far to be available.

Some countries provided themselves of National Code namely: New Mexico (NMAC 14.7.4. 2003 New Mexico Earthen Building Materials Code); India (Indian standard Code of practice for in situ construction of walls in buildings with soil-cement); New Zeland (NZS 4297- 4298-4299, 1998); Perù (NTE E 080,2000); Regional Africa (ARSO 1996); Zimbabwe SAZS 724, 2001).

In other Countries, instead, Guidelines with the validity of Normative documents are presents: Uk (Rammed earth design and construction guidelines); Germany (Lehmbau Regeln ,1999); Australia (HB 195, 2002, Bulletin 5, 1995, EBAA, 2001; Spain MOPT 1992, IETCC cc (1971).


In any case the common tendency is to relate the properties of the initial soil to the performance requirements of the earthen products (Ciancio 20013) in order to define suitability criteria to select and test soil, during the production of rammed earth constructions.

The main suitability criteria concerning the physical properties of the row material are: particle size distribution PSD, Plasticity index, clay content, moisture content, compactability, linear shrinkage.

The main performance requirements are: Compressive strength, flexural strength, drying shrinkage.

Specifically, the compressive strength depends on the dry density. The main factors that influence the density are grain size distribution and the moisture content, since the maximum density is reach when the voids between particles are filled as much as possible.

The water content is defined as the ratio of the mass of water \( M_w \) and the mass of \( M_s \) soil solids

\[
w = \frac{M_w}{M_s} \times 100 \tag{2}\]

Density is distinguished in dry density \( \rho_s \) and wet or apparent density \( \rho \):

\[
\rho_s = \frac{M_s}{V_s} \tag{3}
\]

\[
\rho = \frac{M_w + M_s}{V_t} = \frac{M_t}{V_t} \tag{4}
\]
where:
$M_t$ is the total mass comprehensive of water $M_w$ and solid soil $M_s$;
$V_t$ is the total volume;
$V_s$ is the volume of the solid soil.

The attended values of the used mixture for these two properties are presented in Table 4

\[
\begin{array}{cc}
\rho_s \text{ g/cm}^3 & \rho_{apparent} \text{ g/cm}^3 \\
1.74 & 1.96 \\
\end{array}
\]

Table 4-Density of the soil mixture

The water content is responsible for the plasticity in cohesive soils. Plasticity can be defined as the ability of materials to keep a deformation without breaking (Delgado 2007). It is quantified by the Atterberg limits.

Furthermore clay particles are almost always hydrated in nature, they are surrounded by a layer of adsorbed water. This phenomenon is due to the dipolar molecular structure of the water and the consequent hydrogen bonding that it creates with the clay minerals. Water is electrostatically attracted by clay. The dimension of clay minerals and hence their exposed specific surfaces determines the activity. For the reason seen above, clay soil are particularly sensitive to the moisture.

Typically the drying process of wet soil led to a decrease of volume often accompanied by the appearance of cracks.

3.1.1. Physical characterization

3.1.1.1. Soil grading

Similarly to the concrete, also in rammed earth there is a fraction with a binding role, specifically silt and clay that are cohesive soils, and an aggregate fraction: sand and gravels.

Particle size distribution PSD and moisture content are used in this work as reference for the evaluation of the soil suitability, in accordance with the recommendation prescribe by the quoted documents. Specifically minimum and maximum percentages of contents of the soil fractions (clay, silt, sand, gravel) are given as recommended granularity. In Table 5, the limit value of soil fractions for rammed earth are reported (Delgado et. al.2005):

<table>
<thead>
<tr>
<th>Document</th>
<th>Clay %</th>
<th>Silt %</th>
<th>Sand%</th>
<th>Gravel%</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAZS 724 - 2001</td>
<td>5–15</td>
<td>15–30</td>
<td></td>
<td>Sand+ fine gravel 50-70</td>
</tr>
<tr>
<td>MOPT-1992</td>
<td>5–26</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IETCC 1971</td>
<td>10–40</td>
<td>20–40</td>
<td>10–40</td>
<td>10–20 fine gravel</td>
</tr>
</tbody>
</table>

Table 5 Recommended soil fractions
In the present experimental work the composition in Tab 6 is used for the production of the walllettes in accordance with the IETCC recommendation.

<table>
<thead>
<tr>
<th>Clay+silt %</th>
<th>Small sand</th>
<th>Big sand %</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>45</td>
<td>15</td>
</tr>
</tbody>
</table>

Tab 6 soil fractions

The granulometric curve of the soil mixtures falls, approximately within the field defined by the two curves of admissibility, indicated as optimum for earthen construction techniques by some of the quoted literature references (Figure 12; Figure 13).

3.1.1.2. Moisture content and Compaction

In order to obtain the desired level of density and compressive strength, moisture content and compact energy were designed and tested.

Compaction is intended as the process that brings about an increase in soil density and a decrease in air volume.

According to the Proctor compaction theory the optimum moisture content OMC is the one for which the maximum dry density is obtained for a given compactive effort. (Holts and Covaks 1981).

Compaction curves are obtained increasing the water content for different compactive effort applied on the same type of soil. The curves, experimentally obtained, show that increasing the compactive effort, higher dry density are reached for lower optimum moisture content.

Adding progressively water to the soil means that the clay particles are enveloped by larger layer of water until the water takes the place of the particles, therefore the maximum dry density can't be reached for saturation. This is the reason why after the peak points in the compaction curves, the dry density start decreasing in correspondence of an increase of water content.
The standard compaction curve in Figure 14 is used as reference for the production of the three samples.

![Figure 14: Standard and modified compaction curves for Crosby B.](image)

The Figure 14 shows that the maximum dry density of 1.74 Mg/m³ is reached for a water content of approximately 16%.

The reference Energy effort for the standard test, referred to a soil type passing from a size sieve of 4.75mm is equal to 592 kJ/m³ and it’s is determined as follow:

$$E = \frac{N \times n \times W \times h}{V} = 592 \text{ kJ/m}^3$$  \hspace{1cm} (5)

Where:
- $V$ is the volume of the mould;
- $N$ is the number of blows per layer;
- $n$ in the number of layer;
- $W$ is the hammer weight;
- $h$ is the height of drop.

In the standard Proctor test [ASTM AASTHO BS 1377] materials and instruments are featured as follows: the rammer has a weight of 2.495 kg, it is dropped from the height of 304.88 mm, for 25 times on each of the three layer of soil, the utilized mold has a volume of approximately one liter, diameter 101.60±0.40mm height 161.40±0.50mm.
3.1.2. Compressive and flexural strength

From the same mixture produced for each scaled model wall, five cylindrical sample had been extracted in order to perform unconfined compressive tests.

![a) demoulding cylindrical samples for compressive tests](image1)
![b) cylindrical samples for compressive tests](image2)

![c) moulded samples for flexural characterization](image3)
![d) samples for flexural characterization](image4)

Figure 15-Specimens for mechanical characterization

Specifically, as the UK National guidelines (Appendix A) for rammed earth prescribes, Unconfined compressive strength is obtained by testing cylinders having a height to diameter ratio of 2. The samples are high 9 mm and have a diameter of 4.5 mm (Figure 15-b).

To characterize the samples in flexure and tension three point bending flexural test is performed on rectangular cross section specimens 40mmx40mmx160mm (Figure 15-c, d).

Description of the tests and results are presented hereafter in chapter 4.

The characteristics of the glass fiber mesh are reported in Table 7.

The tensile strength is determined on a sample wide 5 cm.

<table>
<thead>
<tr>
<th>mesh</th>
<th>Tested specimen</th>
<th>maximum traction load</th>
</tr>
</thead>
<tbody>
<tr>
<td>weight 65g/m²</td>
<td>width 5 cm</td>
<td>longitudinal direction 891.8 N</td>
</tr>
<tr>
<td>size of the openings 5mmx5mm</td>
<td>clamps distance 20 cm</td>
<td>transverse direction 1117.2 N</td>
</tr>
</tbody>
</table>

Table 7-Glass fiber mesh properties
3.1.3. **Description of the scaled wall models**

In order to investigate the effectiveness of textile reinforcement in rammed earth, three scaled walls have been designed to be tested under shear and compression in plane.

According to the African code for the rammed earth constructions, a slenderness ratio (width:height) of 1:8 for a wall panel has been chosen, therefore the dimensions of an hypothetic wall panel are: 2.80 m in height and length; 35 cm in thickness.

A scale of 1:5 had been used to reduce the model, consequently, the dimensions of the samples were 56 cm in height and length; 7 cm in thickness. (Fig. 5).

![Figure 16](image)

**a) Unreinforced Sample**  
**b) Externally reinforced sample**  
**c) Internally reinforced sample**

*Figure 16-Designed scaled walls specimens*

Besides one unreinforced sample, to be used as reference for comparisons, other two reinforced samples had been designed. The purpose of testing the reinforced sample was to assess the behavior of glass fibers meshes, both as strengthening system for existing structure, and as reinforcing system for new constructions (Figure 16).

In the first case the fibers had been placed on the opposite surfaces of the cured panel. The adhesion between the wall and glass fiber meshes is given by a clay render in which the meshes are supposed to be embedded (Figure 16-b).

In the second case the reinforcing meshes had been placed in the wet mixture during the moulding process of the wall (Figure 16-c). In order to pursuit this aim, the internally reinforced wall needed to be produced horizontally contrarily to real building technique. Even though with the traditional technique it wouldn’t be immediate the application of this reinforcing system, the objective of this work is to find out if the described one, could be an effective reinforcing system for rammed earth new buildings.

In order to be able to compare the result of the tests, all the three sample had been built following the same procedure and using the same horizontal mould.
3.2. Laboratory phases
The production process of the scaled walls is described in this section.
The main phases were:
- Construction of the mould:
- Preparation of the soil mixture;
- Molding and compaction of the samples;
- Samples demoulding;
- Monitoring of drying period.

3.2.1. Preparation of the mould
For the purpose pointed out above, an horizontal wooden mould (Figure 17) had been designed and assembled.
It was constituted by:
- a portable base able to support vertically the quite slender samples during the curing period;
- a removable frame fixed to the base and a back plate, to confine the wet soil during the moulding process and stabilize the sample during the curing time (Figure 17-f);
- a removable back plate, fixed to the base and to the frame (Figure 17-f), to contain the wet mixture during the moulding and ramming process, to be removed during the curing time;
- a front plate to distribute the compaction load (Figure 17-g).

Every wooden component had been cut by mean of an electric saw, after being clamped to a stable horizontal surface (Figure 17b, c).

![a) Wooden components](image1)
![b) Tools implied](image2)
![c) Cutting of the components](image3)

Figure 17- Construction of the mould

Laterafter, the wooden plates had been screwed one each other to form the mould, except for the front plate (Figure 17-g) that was completely removable and it had to be placed inside the frame in the horizontal position.
d) Assembling by screwing  

e) Assembled mould  

f) ready standing mould  

g) Front plate  

h) Demoulding phase: removal of the back plate  

i) Disassembled mould  

Figure 18 Execution of the mould  

The back plate and the frame, could be easily unscrewed one from each other and both from the base in order to be coupled with a new base and form a new mould (Figure 18-h, i)  

3.2.2. Preparation of the mixture  
Particular care has been paid to the treatment of the clay due its high sensitivity to moisture as seen in 3.1.  
It had been sieved (Figure 19 Clay Sieving operations) and then stored in the hygrometric stable environment of the laboratory for 10 days.  

a) Picking unsieved clay  

b) Raft Clay sieving  

c) Clay Sieving with 2mm sieve diameter  

Figure 19 Clay Sieving operations
For the production of the three scaled walls a total wet density of 2130 kg/m³, had been used as reference, from previous researches carried out by the UPC University in Terrassa (tab. 5), therefore, for a volume of the designed specimen (0.56mx0.56mx0.07m), 46.7 kg of mixture had been required. 49 kg of soil mixture had been produced indeed, in order to be employed also in the production of specimens for mechanical characterization and for moisture content determination as well.

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount [g]</th>
<th>Material</th>
<th>Amount [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>water</td>
<td>2100</td>
<td>water</td>
<td>6652</td>
</tr>
<tr>
<td>clay</td>
<td>5347</td>
<td>clay</td>
<td>16938</td>
</tr>
<tr>
<td>small sand</td>
<td>6016</td>
<td>small sand</td>
<td>19057</td>
</tr>
<tr>
<td>big sand</td>
<td>2005</td>
<td>big sand</td>
<td>6351</td>
</tr>
<tr>
<td>Tot</td>
<td>15468</td>
<td>Tot</td>
<td>49000</td>
</tr>
<tr>
<td>solid mass [g]</td>
<td>13368</td>
<td>solid mass [g]</td>
<td>42347,56</td>
</tr>
<tr>
<td>designed MC %</td>
<td>15,71</td>
<td>designed MC %</td>
<td>15,71</td>
</tr>
</tbody>
</table>

Table 8 Soil mixture composition

First the dry soil fractions are manually mixed (fig. 7). Then, water had been added and mixed together with the soil.
3.2.3. Samples production

In order to verify the suitable moisture content, the hand drop test has been performed (Figure 20-d) in accordance to the New Zealand Code.

An handful of wet mixture has to be dropped on to an hard flat surface from a distance of 1.50 m. If the amount of water is not sufficient, then it will not be possible to form a ball or the handful shatters into a star-shaped pattern of powder. If the water content is excessive, then the lump will not break almost at all. The water content is suitable if the handful breaks into a star-shaped smaller pattern containing several lumps. (NZS 4298: 1998).

In Figure 20-d, the mixture shows an appearance close to the one related to an excessive water content. However, to produce all the samples of wall, the same amount of water had been used in order to be able to compare the results of the tests.

![Figure 21 Moulding and compaction](image)

The wet mixture had subsequently compacted (Figure 21)

The required Impact energy of 592 kJ/m3, seen in 3.1.1.2, corresponds to 13 kJ distributed on the entire volume of the scaled wall specimen. To release the latter degree of energy, 190 impact with a 8 kg hammer from 80 cm of height had been programmed. That means 5 impact in 36 position of the wall surface.

Moreover a static compressive load of 180 kg had been applied, since a uniform compaction wasn’t possible to reach, due to the spread configuration of the soil layer and the high water content.

For each walllette The exact moisture content had been determined, by mean of the oven drying test according to IS: 2720 (Part II) – 1973. The moisture content is defined as the ratio between the solid mass and the mass of the water. The amount of water present in the mixture had been determined by following the steps presented hereafter.
Firstly a clean and dry glass container had been weighted, then, the weight of the container filled with wet mixture was measured. The sample, composed as just described (glass + wet soil), had been left in oven at a temperature of 110° C for 24 hours, subsequently it had been weighted again (Figure 22).

![Figure 22 Dry oven test](image)

Finally the moisture content had been calculate by this relationship:

\[ w = \frac{W_2 - W_1}{W_3 - W_1} \times 100\% \]  

(6)

Where:
- \( w \) is the water content
- \( W_1 \) is the weight of empty container
- \( W_2 \) is the weight of the container with the wet soil
- \( W_3 \) is the weight of the container with the dry soil.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( w ) glass [g]</th>
<th>( w ) glass + wet soil [g]</th>
<th>( w ) glass + dry soil [g]</th>
<th>MC [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W1.EXT</td>
<td>90,3</td>
<td>369,3</td>
<td>330,7</td>
<td>16,057</td>
</tr>
<tr>
<td>RE_W2.INT</td>
<td>92</td>
<td>405,4</td>
<td>365,8</td>
<td>14,463</td>
</tr>
<tr>
<td>RE_W3.UNR</td>
<td>92,8</td>
<td>352,8</td>
<td>224,2</td>
<td>15,97</td>
</tr>
</tbody>
</table>

Table 9-Moisture contents determined by mean of the dry oven test

After being compacted the sample had been turned in vertical position. The demoulding operations (Figure 23) had been led gradually to allow an as fast as possible drying, though avoiding the appearance of deformations due to shrinkage. The black plate is removed two days after the production of the wet mixture, while the frame is removed after one week. The demoulded sample is left drying standing vertically on the base.
Ultrasonic pulse velocity test had been performed in order to monitor the moisture content lowering. Two cylindrical transducers had been acoustically coupled in direct transmission one on each side of the wall. The transit time had been registered in different reference points. Those value decreased as the moisture content decreased.

IN Table 10 are listed the scaled wall models. Identification label and corresponding description are reported. The numbering respect the production order.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W0</td>
<td>Unreinforced low compacted</td>
</tr>
<tr>
<td>RE_W1.EXT</td>
<td>Externally reinforced</td>
</tr>
<tr>
<td>RE_W2.INT</td>
<td>Internally reinforced</td>
</tr>
<tr>
<td>RE_W3.UNR</td>
<td>Unreinforced</td>
</tr>
</tbody>
</table>

Table 10 Specimens identification codes

In the subsequent chapters the abbreviation of the test performed on the specimen it is included in the label (Table 11).

<table>
<thead>
<tr>
<th>Label</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO</td>
<td>Axial compression</td>
</tr>
<tr>
<td>Flex</td>
<td>Three point flexural</td>
</tr>
<tr>
<td>D</td>
<td>Diagonal compression</td>
</tr>
</tbody>
</table>

Table 11-test identification code
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings

- **Sample RE_W0**
  The first produced sample was designed with a water content of 11.22%. It has been roughly compacted by mean of a low static load.
  It is used to proof the stability of the mould and to calibrate the test instruments.

- **Sample 1 RE_W1_EXT**
  The second produced model, hereafter referred to as RE_W1.EXT, was the one designed to be externally reinforced. In spite of the intention to standardize as much as possible the production of the reduced scale model of walls, the manufacturing operations presented inevitably some differences that sometimes could notably affect the characteristics of the products.
  During this process it had been noticed that the consistency of the soil mixture resulted to be too plastic, hence it had been decided to reach only the 14.5% of water content instead of 15.7 as designed, thus, higher than the one that was supposed to be added. The latter phenomenon is probably due to a residual adsorbed water as explained in 3.1.
  Before being reinforced, the sample had been left curing for three weeks.
  The readings of the UPV test are reported in Table 12.

<table>
<thead>
<tr>
<th>Date</th>
<th>Transit time [µs]</th>
</tr>
</thead>
<tbody>
<tr>
<td>22/05/2017</td>
<td>A  48  B  51  C  43  D  58  E  55  F  55</td>
</tr>
<tr>
<td>24/05/2017</td>
<td>A  42  B  45  C  37  D  56.5  E  56.3  F  57.5</td>
</tr>
<tr>
<td>26/05/2017</td>
<td>A  36  B  38  C  33  D  51  E  51  F  50</td>
</tr>
<tr>
<td>29/05/2017</td>
<td>A  33  B  35  C  31  D  41.6  E  46  F  41.4</td>
</tr>
</tbody>
</table>

Table 12- UPV results in sample 1 before being reinforced

From the recorded values (Table 12) of the transit time it can be observed that in the upper part of the specimen the drying had been faster and more uniform than in the lower part (Figure 25).
Moreover the gap between two consecutive readings, progressively decrease until becoming completely stable.

![Figure 25- UPV results in sample 1 before being reinforced](image-url)
The glass fiber mesh is applied on both sides of the wall as shown in Figure 26.

Figure 26 Execution of the external reinforcement on sample RE_W1.EXT

The dried surfaces needed to be moisturized in order to foster the adhesion of the clay plaster in which the mesh are embedded. A first layer of 2 mm circa, had been smeared on the surface of the wall in a rough way to help the penetration of the soil articles inside the fabric of the mesh. The mesh had been fixed by mean of the application of a second layer of plaster starting from the corner and the edges of the wall to avoid wrinkles and movements of the fabric. The outer plaster layer had been subsequently smoothed. The plaster was made of a clay fraction of 70% and a small sand fraction of 30%.

To limit the appearance of cracks due to the drying shrinkage of a so clayey mixture, the wall is covered by a plastic envelope a two bowls of water had been placed next to the wall under the covering, as curing provisions.
A new series of UPV test are carried out in order to check the adhesion of the reinforcement to the wall. (Table 13, Figure 27).

<table>
<thead>
<tr>
<th>Dates</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
<td>08/06/2017</td>
<td>52.5</td>
<td>56.3</td>
<td>56</td>
<td>56.2</td>
<td>57</td>
<td>84</td>
<td>59</td>
</tr>
<tr>
<td>13/06/2017</td>
<td>47.2</td>
<td>43.3</td>
<td>48.2</td>
<td>47.3</td>
<td>46.6</td>
<td>47.2</td>
<td>51.6</td>
</tr>
<tr>
<td>19/06/2017</td>
<td>43</td>
<td>41.1</td>
<td>41.8</td>
<td>44.1</td>
<td>43.1</td>
<td>43.7</td>
<td>48.6</td>
</tr>
<tr>
<td>21/06/2017</td>
<td>43.3</td>
<td>40.2</td>
<td>43.9</td>
<td>43</td>
<td>42.4</td>
<td>43.6</td>
<td>49.3</td>
</tr>
<tr>
<td>23/06/2017</td>
<td>43.1</td>
<td>40.2</td>
<td>43.9</td>
<td>43</td>
<td>42.4</td>
<td>43.6</td>
<td>49</td>
</tr>
</tbody>
</table>

Table 13 UPV results in sample 1 after the reinforcement

Several reference points are taken. From these reading results that the reinforcement is quite good attached from a distance of 10 cm from the edges. Out of this limit there was no transmission of sonic waves due to discontinuities in the medium.

![Figure 27 - UPV results in sample 1 after the reinforcement](image)

- **Sample RE_W2.INT**

The scaled wall indicated as sample 2 had been internally reinforced with the insertion of two mesh inside 0.5mm from the outer wall surface.

After the mixing, a first amount of soil 3.3kg. had been placed in the mould. Subsequently the earth had been manually mixed to receive the fiber mesh. The remaining amount of soil had been poured into the mould, except for the quantity earmarked to the production of the samples for the compressive characterization, besides the same quantity of the outer layer of soil that has to cover the second mesh.
During the production of this sample, the wet soil had been mixed by mean of an electric mixer. Moreover a lower compaction effort in term of ramming impact had been applied. Probably these are the causes of the lumpy texture of the mass. The UPV test didn’t give any result, hence, as is visible, there is no connection between the layers defined by the meshes.
Sample RE_W3.UNR

The last sample to be produced is the unreinforced one, RE_W3.UNR. As following, the ultrasonic test readings are reported (Table 14, Figure 28)

<table>
<thead>
<tr>
<th>Dates</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>13/06/2017</td>
<td>53.2</td>
<td>59</td>
<td>54.3</td>
<td>60.3</td>
<td>66</td>
<td>72.8</td>
</tr>
<tr>
<td>19/06/2017</td>
<td>40</td>
<td>42.4</td>
<td>40.3</td>
<td>46.4</td>
<td>45.4</td>
<td>38.8</td>
</tr>
<tr>
<td>23/06/2017</td>
<td>38.3</td>
<td>40.1</td>
<td>38.2</td>
<td>44.3</td>
<td>42.2</td>
<td>36.2</td>
</tr>
<tr>
<td>26/06/2017</td>
<td>38.6</td>
<td>39.6</td>
<td>39.3</td>
<td>41.5</td>
<td>39.7</td>
<td>35.5</td>
</tr>
</tbody>
</table>

Table 14- UPV test result in the unreinforced sample

It has to be noticed that for the last produced specimen, the drying process was faster since it took place later in summer season.
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings
4. Experimental Tests

It was decided to test the scaled wall model RE_W2 INT, internally reinforced, and the corresponding cylindrical samples, since it had been drying for more than three weeks. Thereafter, the sample RE_W1 EXT had been tested as soon the measurement of the ultrasonic test were stable, hence the external reinforced plaster was completely dry. Specifically, the sample RE_W1 EXT had been testes after 20 days from the reinforcement placing and 5 weeks after the demoulding. Finally, the unreinforced sample, RE_W3 UNR had been tested after three weeks of curing.

4.1. Mechanical characterization

In order to characterize the material from the mechanical point of view, unconfined compressive tests and three points flexural test had been performed on samples made with the same soil mixture of the scaled wall models.

4.1.1. Unconfined compressive test.

For each scaled wall, the unconfined compressive strength is determined, as recommended by the UK National guidelines for rammed earth. Five cylindrical samples had been produced from the same mixture used for the wall. The Uk national guidelines prescribe that the height of the cylinder has to be the double of the diameter. The cylindrical specimen underwent to compression along their longitudinal axis in order to determine the unconfined compressive strength. The latter is defined as the ratio between the maximum load and the cross section of the sample. Moreover, for each sample, the compressive strength is associated with the dry density. In accordance with the British guidelines, the unconfined characteristic compressive strength $f_c$ is determined as in the equation 6:

$$f_c = f_a - 1.65\sigma_{n-1}$$

(6)

Where: $f_a$ is the average unconfined compressive strength of test sample; $\sigma_{n-1}$ is the standard deviation of test sample.

Figure 29 - Unconfined compressive test
- Specimen RE_W2.INT

The first sample to be tested was the ones produced with the mixture of the internally reinforced sample. In Table 1 are presented the geometrical characteristics of the cylindrical samples and the corresponding values of the dry densities.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>W [g]</th>
<th>d [cm]</th>
<th>h [cm]</th>
<th>V cm³</th>
<th>density [g/cm³]</th>
<th>[Kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W2.INT_CO1</td>
<td>217,8</td>
<td>4,2</td>
<td>7,7</td>
<td>106,7</td>
<td>2,0</td>
<td>2041,6</td>
</tr>
<tr>
<td>RE_W2.INT_CO2</td>
<td>220,6</td>
<td>4</td>
<td>8,1</td>
<td>101,8</td>
<td>2,2</td>
<td>2167,3</td>
</tr>
<tr>
<td>RE_W2.INT_CO3</td>
<td>207,8</td>
<td>4</td>
<td>7,6</td>
<td>95,5</td>
<td>2,2</td>
<td>2175,8</td>
</tr>
<tr>
<td>RE_W2.INT_CO4</td>
<td>223</td>
<td>4,2</td>
<td>8</td>
<td>110,8</td>
<td>2,0</td>
<td>2012,0</td>
</tr>
<tr>
<td>RE_W2.INT_CO5</td>
<td>215,3</td>
<td>4</td>
<td>7,8</td>
<td>98,0</td>
<td>2,2</td>
<td>2196,5</td>
</tr>
</tbody>
</table>

Average: 2,1
dev: 0,1
cov %: 4,0

Table 15- Density RE_W2.INT

In Table 16 are presented for each specimen, the values of the maximum load resulted from the uniaxial compressive test and the corresponding compressive strength.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cross section area [mm²]</th>
<th>F max N</th>
<th>f [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W1_CO_1</td>
<td>1385</td>
<td>946</td>
<td>0,7</td>
</tr>
<tr>
<td>RE_W1_CO_2</td>
<td>1257</td>
<td>920</td>
<td>0,7</td>
</tr>
<tr>
<td>RE_W1_CO_3</td>
<td>1257</td>
<td>679</td>
<td>0,5</td>
</tr>
<tr>
<td>RE_W1_CO_4</td>
<td>1385</td>
<td>1289</td>
<td>0,9</td>
</tr>
<tr>
<td>RE_W1_CO_5</td>
<td>1257</td>
<td>368</td>
<td>0,3</td>
</tr>
</tbody>
</table>

Average: 0,6
dev: 0,2
cov %: 37,3

Table 16 Compressive strength RE_W1_CO

The unconfined characteristic compressive strength for this specimen is \( f_c = 0,24 \text{ N/mm}^2 \).

In fig. the load displacement curves are provided. It can be noticed that the development of the tests is quite anomalous and irregular, probably, due to the set-up apparatus, hence not all of the five test are reliable and it would be better to consider only the result that show lower variance coefficients.
• Specimen RE_W1.EXT

The same procedure had been followed for the samples of the externally reinforced scaled wall model. Physical and geometrical properties are presented in the tab.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>W [g]</th>
<th>d [cm]</th>
<th>h [cm]</th>
<th>V [cm³]</th>
<th>density [g/cm³]</th>
<th>density [Kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W1_CO_1</td>
<td>237,7</td>
<td>4,5</td>
<td>8,5</td>
<td>135,2</td>
<td>1,8</td>
<td>1758,3</td>
</tr>
<tr>
<td>RE_W1_CO_2</td>
<td>223,5</td>
<td>4</td>
<td>8</td>
<td>100,5</td>
<td>2,2</td>
<td>2223,2</td>
</tr>
<tr>
<td>RE_W1_CO_3</td>
<td>219,6</td>
<td>4</td>
<td>8</td>
<td>100,5</td>
<td>2,2</td>
<td>2184,4</td>
</tr>
<tr>
<td>RE_W1_CO_4</td>
<td>219</td>
<td>4</td>
<td>8</td>
<td>100,5</td>
<td>2,2</td>
<td>2178,4</td>
</tr>
<tr>
<td>RE_W1_CO_5</td>
<td>225,9</td>
<td>4,1</td>
<td>8,3</td>
<td>109,6</td>
<td>2,1</td>
<td>2061,5</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2,1</td>
<td>2081,2</td>
</tr>
<tr>
<td>dev</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0,2</td>
<td>190,3</td>
</tr>
<tr>
<td>cov %</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9,1</td>
<td></td>
</tr>
</tbody>
</table>

For this sample the results of only two specimens are presented, since the remaining three are not interpretable and increase too much the variance coefficient.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cross section area [mm²]</th>
<th>F max [N]</th>
<th>f [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W1_CO_1</td>
<td>1392</td>
<td>845</td>
<td>0,61</td>
</tr>
<tr>
<td>RE_W1_CO_3</td>
<td>1343</td>
<td>1054</td>
<td>0,78</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td>0,70</td>
</tr>
<tr>
<td>dev st</td>
<td></td>
<td></td>
<td>0,13</td>
</tr>
<tr>
<td>cov</td>
<td></td>
<td></td>
<td>18,07</td>
</tr>
</tbody>
</table>

The characteristic compressive strength is **0,48 N/mm²**

Figure 31-Load Displacement curves RE_W1.INT
The characteristic compressive strength is $f_{c}=1.57 \text{ N/mm}^2$. 

Figure 32-Load displacement curves of compression test on RE_W3.UNR
The latter results can be considered the most reliable ones since the compressive effort is similar the one applied during the production of the scaled wall.

The values of stiffness and Young modulus are determined as in eq. and reported in tab.

\[
K = \frac{P}{\delta} \quad (7)
\]
\[
E = \frac{Kh}{A} \quad (8)
\]

Where:
- \(P\) is the axial load implied to provoke the displacement \(\delta\) considered between the 30% and the 70% of the maximum load;
- \(A\) is the cross section area of the cylindrical specimen;
- \(h\) is the height of the specimen.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(K) [N/mm]</th>
<th>(E) [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W3_CO_1</td>
<td>3010</td>
<td>205</td>
</tr>
<tr>
<td>RE_W3_CO_2</td>
<td>2354</td>
<td>161</td>
</tr>
<tr>
<td>RE_W3_CO_3</td>
<td>2556</td>
<td>194</td>
</tr>
<tr>
<td>RE_W3_CO_4</td>
<td>3061</td>
<td>210</td>
</tr>
<tr>
<td>RE_W3_CO_5</td>
<td>2171</td>
<td>152</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>2630</strong></td>
<td><strong>184</strong></td>
</tr>
<tr>
<td><strong>dev</strong></td>
<td><strong>395</strong></td>
<td><strong>26</strong></td>
</tr>
<tr>
<td><strong>cov %</strong></td>
<td><strong>15</strong></td>
<td><strong>14</strong></td>
</tr>
</tbody>
</table>

Table 18 Stiffness and apparent young modulus RE_W3.UNR_CO

The young modulus has to be intended as apparent modulus, since the bulk behavior of the material is not known.
4.1.2. Three point bending test

The three point bending test was performed in order to determine the flexural strength and the compressive strength of the mixture used both for the plaster of the externally reinforced scaled wall, and for the one used for the walls as well.

As references for the test procedure the standard UNI EN 1015_11 was adopted.

The three points bending test was carried out with a rate of 10 N/s of force control.

On the two half of the prismatic samples obtained, compressive test were carried out under force control with a rate of 100 N/s.

![Three point bending test](image)

Figure 33-a),b) Three point bending test; c),d) Compression test

The flexural strength is calculated as in the equation (9)

\[
f = 1.5 \frac{F l}{bd^2}
\]

Where: F is the maximum load

l is the distance between the two supporting points

b is width of the prism

d is high of the prism

**RE_W1.INT Plaster**

The initial dimensions of the prismatic sample were the one presented in the Table 1919

<table>
<thead>
<tr>
<th>b_i [mm]</th>
<th>d_i [mm]</th>
<th>L_i [mm]</th>
<th>V_i [mm^3]</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>40</td>
<td>160</td>
<td>256000</td>
</tr>
</tbody>
</table>

Table 1919: Dimensions of the Plaster prismatic sample

At the moment when the test were performed, the sample presented the value of linear shrinkage provided in the Table 20:

<table>
<thead>
<tr>
<th>Specimen</th>
<th>L [mm]</th>
<th>linear shrinkage %</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLA:flex:1</td>
<td>149.34</td>
<td>6.66</td>
</tr>
<tr>
<td>PLA:flex:2</td>
<td>149.73</td>
<td>6.42</td>
</tr>
<tr>
<td>PLA:flex:3</td>
<td>149.66</td>
<td>6.46</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>6.51</strong></td>
</tr>
<tr>
<td>dev st</td>
<td></td>
<td>0.13</td>
</tr>
<tr>
<td>Cov %</td>
<td></td>
<td>1.99</td>
</tr>
</tbody>
</table>

Table 20: Linear shrinkage of the plaster
The values of the flexural strength, obtained by mean of the equation (9) are presented in Table 21.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>b [mm]</th>
<th>d [mm]</th>
<th>l [mm]</th>
<th>F [N]</th>
<th>f [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLA_flex_1</td>
<td>37,53</td>
<td>37,37</td>
<td>100</td>
<td>274</td>
<td>0,78</td>
</tr>
<tr>
<td>PLA_flex_2</td>
<td>37,72</td>
<td>38,63</td>
<td>100</td>
<td>309</td>
<td>0,82</td>
</tr>
<tr>
<td>PLA_flex_3</td>
<td>37,7</td>
<td>38,52</td>
<td>100</td>
<td>270</td>
<td>0,72</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>0,78</strong></td>
</tr>
</tbody>
</table>

Table 21 - Flexural strength of the plaster

Moreover, the load displacements curves are provided in the graphs in Figure 34.

![Flexure test Plaster](image)

Figure 34 - Flexural test curves of the plaster

For each of the two portion obtained from the three point bending test, the compression test was carried out (Errore. L'origine riferimento non è stata trovata.).

![Flexure test Plaster](image)

Figure 34 - Flexural test curves of the plaster

The sample was placed between two aligned square plates, of 40 mm per side, one at the base and one at the top (Figure 33 c,d). Here after the result of the compression tests are presented:
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings

Figure 35-Compression test on plaster samples

The variance coefficient highlight the difference between the values of compressive strength, moreover, as it ’s clear from the load displacement curves, for the two half of the same specimen, the evolution of the compressive test it are quite far one from the other.

• RE_W3.UNR

The three points bending test was performed also on three prismatic sample having the same composition of the unreinforced scaled wall RE_W3.UNR.

The results are presented in the graphs and tables below:

<table>
<thead>
<tr>
<th>Specimen</th>
<th>L [mm]</th>
<th>Linear Shrinkage %</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W3.UNR_flex_1</td>
<td>155</td>
<td>3,13</td>
</tr>
<tr>
<td>RE_W3.UNR_flex_2</td>
<td>156</td>
<td>2,50</td>
</tr>
<tr>
<td>RE_W3.UNR_flex_3</td>
<td>156</td>
<td>2,50</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>2,71</strong></td>
</tr>
<tr>
<td>dev</td>
<td></td>
<td>0,36</td>
</tr>
<tr>
<td>Cov %</td>
<td></td>
<td>13,32</td>
</tr>
</tbody>
</table>

Table 23 Linear shrinkage of the unreinforced specimen

The linear shrinkage and the flexural strength are inferior than the one of the plasters’ prismatic specimen that were richer in water and clay.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>b [mm]</th>
<th>d [mm]</th>
<th>l [mm]</th>
<th>F [N]</th>
<th>f [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W3.UNR_flex_1</td>
<td>39,24</td>
<td>39,23</td>
<td>100</td>
<td>176</td>
<td>0,43</td>
</tr>
<tr>
<td>RE_W3.UNR_flex_2</td>
<td>39,58</td>
<td>40,24</td>
<td>100</td>
<td>237</td>
<td>0,55</td>
</tr>
<tr>
<td>RE_W3.UNR_flex_3</td>
<td>39,57</td>
<td>40,09</td>
<td>100</td>
<td>202</td>
<td>0,48</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>0,50</strong></td>
</tr>
<tr>
<td>dev</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0,06</td>
</tr>
<tr>
<td>Cov %</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12,22</td>
</tr>
</tbody>
</table>

Table 24-Flexural strength of the unreinforced sample
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings

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ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Figure 36 - Flexural strength of the unreinforced specimen

Also in this case, compressive test were carried out on the two halves of the prisms. The values of compressive strength obtained and the curves load displacement related to the test are presented in Table 25 - Compressive strength RE_W3.UNR on prismatic samples

<table>
<thead>
<tr>
<th>Specimen</th>
<th>F [N]</th>
<th>fc [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W3.UNR_CO1_A</td>
<td>1816</td>
<td>1,13</td>
</tr>
<tr>
<td>RE_W3.UNR_CO1_B</td>
<td>2210</td>
<td>1,38</td>
</tr>
<tr>
<td>RE_W3.UNR_CO2_A</td>
<td>3100</td>
<td>1,93</td>
</tr>
<tr>
<td>RE_W3.UNR_CO2_B</td>
<td>1855</td>
<td>1,16</td>
</tr>
<tr>
<td>RE_W3.UNR_CO3_A</td>
<td>2513</td>
<td>1,57</td>
</tr>
<tr>
<td>RE_W3.UNR_CO3_B</td>
<td>2508</td>
<td>1,57</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>1,46</td>
</tr>
<tr>
<td><strong>dev</strong></td>
<td></td>
<td>0,30</td>
</tr>
<tr>
<td><strong>cov %</strong></td>
<td></td>
<td>20,67</td>
</tr>
</tbody>
</table>

Table 25 - Compressive strength RE_W3.UNR on prismatic samples

The compressive strength obtained on prismatic samples tested under force control is 7% lower than the one obtained from the cylindric samples tested under displacement control, hence they are quite the same. The standard variation in the latter test, though, is 20% against the 11% on the compression test performed on the cylindrical samples.

Figure 37 - Compression test on RE_W3.UNR prismatic samples
4.2. In-plane cyclic shear-compression

The main aim of this work is to evaluate the in-plan behavior of reinforced rammed-earth wall, subjected to seismic action. The horizontal dynamic thrust is simulated by a cyclical shear action, while the vertical axial compression, due to the weight is supposed to be constant.

To assess the effectiveness of the two different reinforcement systems, and compare the behavior of the different model configurations under the same load condition, all the scaled walls were initially designed to be tested with the same set-up for the in-plane cyclic shear-compression. The latter revealed to be inappropriate, hence, diagonal compression tests were decided to be performed.

4.2.1. Test set-up

To assess the in plane behavior of the scaled walls, the latter had been tested under shear and compression. The samples were placed on a support that allows the horizontal translation in the shear plan direction, whereas the same movement was restrained at the top of the wall.

The cyclical shear load was transmitted at the free edge by mean of an horizontal actuator, while a constant axial compression was applied at the top of the. The actuator at the base, transmitted the shear by mean of an horizontal frame, made of aluminum profiles, that was surrounding the perimeter of the wall. Similarly, another horizontal frame, fixed at the rigid steel frame of the testing apparatus, avoided the same movement at the top of the wall.

A C steel profile and wooden elements had been used to distribute the compressive load. This set-up configuration had been utilized for testing the calibration sample RE_W0 and for the sample internally reinforced RE_W2_INT.

![In-plane shear compression test set-up](image)

The two hydraulic actuators were connected to a controller and monitored by two sensors, one for the load and one for the displacements.
4.2.2. Test execution and outcomes

The test procedure is performed following the standard ASTM E 2126 – 02°, method B as reference. The shear action starts as soon the vertical load reaches the value of 5 kN that is kept constant during the test. Horizontal displacements are applied with increasing amplitude each three cycles per step as in the pattern showed in Table 26- Amplitudes of the reversed cycles.

<table>
<thead>
<tr>
<th>step</th>
<th>cycles</th>
<th>Amplitude, % Δu(=20mm)</th>
<th>Δd [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>1.25</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
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<tr>
<td>4</td>
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</tr>
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<td>10</td>
<td>2</td>
</tr>
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<td>6</td>
<td>3</td>
<td>20</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>40</td>
<td>8</td>
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<tr>
<td>8</td>
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<td>14</td>
<td>3</td>
<td>180</td>
<td>36</td>
</tr>
<tr>
<td>15</td>
<td>3</td>
<td>200</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 26- Amplitudes of the reversed cycles

The actuator displacement was controlled at a constant cyclic frequency of 0.2 Hz. The ultimate displacement had been assumed at 20 mm. A first test was firstly performed on the sample RE_W0 in order to calibrate the test apparatus. An ultimate displacement of 40mm had been reached until the local damages at the base and partial failure for compression at the top occurred(Figure 39-in-plan shear compression test on RE_W0).

Figure 39-in-plan shear compression test on RE_W0
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings

It wasn’t possible to individuate a clear crack pattern. Only a vertical crack due to vertical compression is visible. The damages mostly appears locally at the base where the load is applied and at the top corner where the restrains were applied. The described type of damages were due to the extremely incoherent consistence of the material. Moreover the asymmetrical distribution of the damages were due to an incorrect insertion of the input data. From the graphs on Figure 40, it’s possible to notice that after each cycle, the new starting point for the application of the next step with amplified displacement is the last of the previous step, hence it needs to be brought again to the zero level.

In fig. the development of horizontal displacement and horizontal load versus time and hysteretic loop are reported. Here it’s clear the addition of displacement and load only in one direction, starting from the last value of the third cycle of each step.

Figure 40-In-plane cyclical loading test: a) Cycle displacement schedule; b) load pattern; c) hysteretic loops
Experimental study on the influence of textile reinforcement on the structural response of reduced scale rammed earth buildings

- **Specimen RE_W2.INT**
  The same test had been performed on the sample RE_W2_INT. The scaled wall was designed and produced to have two layers of glass fiber reinforcement.
  As it described in the 3.2.3, this sample showed an incoherent and uneven consistence since as soon it had been demoulded. The UPV test proved, moreover the presence of discontinuity and voids in the thickness of the wall.
  During the operation of placing the sample on the testing apparatus, one external layer of dry soil, completely detached from the fiber mesh. That means that the glass fiber mesh didn't allowed at all the penetration of the wet soil particles in the bottom layer of the horizontal sample, hence the less compacted. The latter phenomenon could have been generated due to several causes namely:
  - Insufficient compactive effort;
  - High moisture content that increased the internal cohesion of the wet soil clumps more than the external adhesion inside the whole mass, probably increased by the use of the automatic kneading.
  - Excessively small spacing of the glass fiber mesh to allow the penetration of the soil mixture particles.

The in plane compression shear-cyclical test had been carried out with the same set-up and load pattern as for the sample RE_W0.

The input data were adjusted so that each cycle start from zero.

In the figure, the Cycle Displacement Schedule, the load pattern and the hysteresis curves are presented in the Figure 41.

The cycle displacement schedule shows that increments in the displacement amplitude occur every three loading cycles.

![Figure 41 In-plane cyclical behavior on RE_W2.INT: a) Cycle displacement; b) Load pattern](image)

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The hysteresis loops and, more clearly the envelope curve, shows a fall in the load values after the first peak and a successive recovering branch towards a new lower peak.

A similar behavior, for the same sample, occurs also in different type of test hereafter presented.

![Hysteresis curves](image1)

![Envelope curve](image2)

**Figure 42- In-plane cyclical behavior on RE_W2.INT: a) Hysteresis curves: b) Envelope curve**

In spite of the compressive load and the ultimate displacement applied, also in this case the failure didn’t occur due to shear, whereas only local damage are the corner appeared.

![Damage pattern](image3)

**Figure 43-damage pattern on RE_W2.INT after the cyclical shear test**

Particularly, it has to be noticed that the local crashing in the base corners, where the shear loads are applied, caused the detachment of the external layer of the specimen that was supposed embedding the glass fiber mesh (Figure 43).

### 4.3. In-plane monotonic shear-compression test

In order to pursue the shear behavior, a monotonic test was establish to be performed until failure. The static in plane shear test, was carried out with the same set-up of the cyclical test.

A vertical load of 15 kN was applied during the entire test, an ultimate lateral displacement of 14,5 mm, was reached before that the test was stopped. The maximum lateral load reached was equal to 28,13 kN.
The curve load displacement (Figure 44) shows a fall in the load development followed by a new increasing branch towards a new peak as well as in the cyclical test.

![Monotonic shear test RE_W1.INT](image)

Figure 44-Load displacement curve of the monotonic shear test on RE_W2.INT

The only observed failure mode was the intensification of the previous local damages, for this reason the static test was terminated. It wasn’t possible, neither in this case to observe the failure behavior in shear nor a clear crack pattern. Subsequently, it had been decided to change test program, hence diagonal compressive test were established to be performed.

4.4. Diagonal Compression test

The aim of the diagonal compression test is to measure the diagonal tensile strength of masonry, indirectly generated by compressive stress.

4.4.1. Test set-up

The very first set up for the diagonal compressive test, was quite essential because it had been arranged very quickly, as soon as it had been realized that the previous test was ineffective, In order to carry it out on the sample RE_W2_INT (internally reinforced) that had already undergone to cyclical and static shear test. It had been possible to use the same sample for different test, since the damage experienced, effected locally the corners of the scaled model.

Two adjacent triangular steel profile were clamped together to constitute the base for the rectangular sample diagonally placed. The vertical compressive axial load was transmitted by a triangular steel profile to the sample. The hydraulic actuator was controlled by displacement at a rate of 10 mm/min. Some improvement had been brought to test set-up in the successive tests. Specifically in order to avoid the out of plane overturning for the test RE_W1.INT_D1, two coupled steel profile had been placed vertically around on opposite faces of the loaded corner. The supported corner at the base was clamped by mean of wood elements, steel profiles and F clamps in the tests RE_W3.UNR_D2; RE_W1.INT_D2.
The two set-up are illustrated in Figure 45 and Figure 46. The fist one will be referred as D1, while the second one, provided of restrains at the top and the bottom corner, is identified as D2.
4.4.2. Test execution and outcomes

- Specimen RE_W2.INT: Test set-up D1

The Diagonal compression test was firstly performed on the internally reinforced scaled wall, already tested in in-plane shear. The failure mode was mainly due to crushing of the loaded corner.

Fractures parallel to the axial plane developed mainly in the thickness, with late and slight effects on the external surfaces (Figure 457, b). The glass mesh fiber defined a plane of detachment for the external layer of the sample, although it remained attached to the latter one. Finally the sample underwent to a complete out-of-plane overturning (Figure 457, c).

The maximum load registered is 10565.13 N. The load versus displacement curve, presents a vertical fall of the load after a first and higher peak, followed by a recover and another lower peak. A similar behavior has been observed in the in-plane shear test both cyclical and monotonic as well. This phenomenon is probably caused by a redistribution of the load from the core of the sample to the reinforced layer as the sequence of the failure mode shown in Figure 47.
Specimen RE_W1.EXT: Test set-up D1

The reinforced sample RE_W1.EXT was tested with the set-up D1, except for some measures taken to prevent the out of plane overturning of the quite slender sample.

![Image of test setup](image1)

![Image of test specimen](image2)

![Image of test specimen](image3)

Figure 49-Diagonal compression test performed on the specimen RE_W1.EXT with the set-up D1

The observed failure behavior involved, once again, the crushing of the loaded corner, furthermore it is characterized by the delamination of layers parallel to the axial plane of the sample, and the local detachment of the fiber reinforced plaster.

![Graph of load-displacement](image4)

Figure 50-Load-displacement curves of the diagonal test D1 performed on the sample RE_W1.EXT

The test was performed two times, for technical problems. The first time that it was performed it showed higher maximum load and higher stiffness given by the slope in the load-displacement curve. The maximum load reached during the first test RE_W1.EXT_D1_I is equal to 19 kN. In the second one RE_W1.EXT the maximum reached load of 13.2 kN remained almost constant until the interruption of the test. The latter was arrested since a clear shear behavior had not been observed.
• Specimen RE_W3.UNR, Test set-up D2

The second set-up used for the diagonal compression test foresaw that the loaded and the supported corner were clamped, in order to avoid premature local damage, due to the excessive fragility of the material.

![Image of the test setup](image1)

During the test procedure, vertical cracks appeared and grew throughout the specimen from corner to corner even though the shape at the top corner is influenced by the tools used to clamp it. Soon after the removal of all the constrains portions of the sample, two portions of the sample separated one from the other along the deep cracks, thus the effective failure mode due to diagonal compression is achieved.

![Image of the test results](image2)

The maximum load registered during the test is 25,181 kN, higher than the one reached for the internally reinforced sample, with a different test set-up as seen before. In this case, the curve exhibits a first peak lower and shorter than the one seen for the internally reinforced sample. Moreover the recovering branch after the fall is almost parallel to the initial loading branch, that means that there isn't any variation in the stiffness.
- **Specimen RE_W2.INT, test set-up D2**

  The diagonal test, performed on the unreinforced scaled wall model, revealed to be effective and suitable for testing the specimen object of the present work. The same set-up, that specifically is provided of clamping systems at the supported and at the load corner, therefore, was implied to test again the externally reinforced sample RE_W1.EXT. The latter one had been already undergone to the diagonal test D1.

  The specimen was placed in the apparatus to be compressed along the diagonal axis perpendicular to the one along which the same test was performed with the previous set-up.

  Figure 53-Failure mode RE_W1.INT_D2

  The failure mode is characterized by severe crushing at the upper edge that shows sharp edges typical in compression failure. Delamination failure mechanism due concentrated loads, characterize this failure mode.

  Small and few vertical cracks on the two surfaces of the sample are revealed under the glass mesh fiber.
The test is interrupted since big deformation are reached under the maximum load equal to 35kN, that, nonetheless it’s the highest reached among all the tests.

Figure 54- load- displacement curve diagonal test D2 on RE_W1.EXD

The reinforced plaster is completely detached only after the removal of the restraints. Moreover, the displacement-load curve confirm that ductility and dissipated energy are visibly increased. Subsequently, It’s possible to infer that the reinforcement redistributed the stress, and increased strength and ductility.

4.4.3. Results Comparison

As follows, the load displacement curves related to each one of the two diagonal test set-up are compared.

In Figure 55 are gathered the curves of the first set-up of diagonal test. This set-up was characterized by the absence of constrains in out of plane direction on the specimens that underwent trough out of plane overturning.

Higher stiffness and higher load carrying capacity were achieved by the externally reinforced sample, when it was tested the first time in diagonal compression test with the second set-up. It has to be
noticed that probably the maximum load is influenced by technical defect. The curve presents a softening branch.

Subsequently, the second test performed on the same specimen, shows a lower stiffness and a lower load capacity, whereas the curve development is clearly bilinear. The degradation in the stiffness between the two tests, performed on the same specimen, is due to the presence of damages.

Similar initial stiffness, conversely, are shown by the curves associated to the diagonal test performed on the externally reinforced sample, that had been already tested in the racking test, and the one related to the diagonal test performed for the second time on the RE_W1.EXT sample. The latter described phenomenon is probably due to the damage state.

Moreover, the second linear branch in the RE_W2.INT, could indicate a redistribution of the stresses in the reinforcement, that is not noticeable for the externally reinforced scaled wall.

<table>
<thead>
<tr>
<th>Diagonal compression test</th>
<th>Max Load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W2.INT_D1</td>
<td>11</td>
</tr>
<tr>
<td>RE_W1.EXT_D1_I</td>
<td>19</td>
</tr>
<tr>
<td>RE_W1.EXT_D1_III</td>
<td>13</td>
</tr>
<tr>
<td>RE_W3.UNR_D2</td>
<td>25</td>
</tr>
<tr>
<td>RE_W1.EXT_D2</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 27-Maximum loads reached in the diagonal compression test by each sample

Looking at the values of the carrying capacities in Table 27 it’s possible to evaluate the reliability of all test procedure performed. Higher values are obtained for the second set-up of the diagonal compression test also for the unreinforced specimen with respect to the internally and externally reinforced samples tested with the first set up. Thus, clamping the edge corners in the out of plane direction definitely contributed to channel the tensile stress towards the center of the wallets. The highest value is obtained for the externally reinforced specimen tested in the second set-up.

Moreover, comparing the load-displacements curves, also the ductility increases greatly in the case of the externally reinforced specimen. The initial stiffness, intended as the slopes of the linear branches
of the two curves are very close in the cases, hence the material had the same behavior in the elastic phase in both specimens. The latter can be assumed as a proof of reliability of the tests and a validation of the results.

Furthermore, the estimated dissipation energy, approximately determined as the area below the load-displacement curves, related to the externally reinforced sample diagonally compressed in the second set-up is 2.32 J, thus five times the one dissipated during the same test performed on the unreinforced sample equal to 0.454 J.

Regarding the behavior of the Internally reinforced scaled wall model, it remarkable to notice that all the load displacement curves of the testes that has been performed on it (Figure 1), exhibit a steep soon after the maximum load reached, subsequently a linear upward branch follows.

The described Phenomenon could be interpreted as the contribution of the reinforcement to the performance of the sample under compression. Specifically the resistance of the reinforced layer is activated consequently to development of high strain values, after that the strength of the unreinforced portion is exceed.
4.5. Interpretation of the tests
As seen in the previous paragraph, the shear performance of the specimens tested with the In-plane cyclic shear-compression procedure, was characterized by damages localized in the edge, due to the low cohesiveness of the material. In this condition, the Navier hypothesis are no longer verified. In order to eventually achieve quantitative result in term of mechanical properties and expected enhancement of the latter ones, some improvements and correction to the implied procedures could be undertaken as described in the next chapters. Moreover, if the shear failure would be achieved shear characteristics could have been determined. The ASTM standards E 2126 – 02, prescribe that at least two identical walls have to be tested in the plane shear compression in to determine the shear strength as follows:

\begin{align*}
V_{\text{peak}} &= \frac{P_{\text{ave peak}}}{L} \quad (10)
\end{align*}

Where:

- \( V_{\text{peak}} \) is the maximum shear strength
- \( P_{\text{ave peak}} \) is the maximum lateral load;
- \( L \) : is the length of the wall.

Moreover, assuming the rammed earth as as isotropic and homogeneous material, in the formulation proposed by Turšnek Čačovič for masonry, the shear strength is determined as referential tensile strength \( f_t \). Equation (10)

Specifically, A wall panel, subjected to in plane compression and lateral loads, experiences a shear failure characterized by diagonal cracks when the principal tensile stresses \( \sigma_t \) exceeds the diagonal tensile strength in correspondence of the maximum lateral load.

\begin{align*}
f_t = \sigma_t &= \sigma_t = \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + (b(\tau_{\text{max}}))^2 - \frac{\sigma_0}{2}} \quad (11)
\end{align*}

Where \( \tau_{\text{max}} \) is the average shear stress in the horizontal section of the wall at the attained maximum horizontal load \( H_{\text{max}} \);

- \( \sigma_0 \) is the average compressive stress in the horizontal section \( A_w \) of the walls due to constant vertical load \( N \):

\begin{align*}
\sigma_0 &= \frac{N}{A_w} \quad (12)
\end{align*}

is an index of the ratio between the height and the length of the wall

The wall is assumed to behave like a cantilever, clamped at the base and loaded at the top.

In the ASTM E 2126 – 02 the secant shear modulus calculated as:

\begin{align*}
G' &= \frac{P}{A} * \frac{H}{L} \quad (13)
\end{align*}
Where:

\( G' \) is the shear modulus of the wall obtained test, defines the secant to the force displacement curve at specified wall displacements; 
\( P \) is the lateral shear force measured at the top edge of the wall; 
\( \Delta \) is the displacement of the top edge of the wall. 

\( H \) and \( L \) are the height and the length of the shear wall respectively. 

Furthermore Tomaževič (Tomaževič 1996) proposes the degradation of secant stiffness as factor to be evaluated during the cyclical evolution of the test:

\[
K = \frac{H}{\Delta}
\]

The secant stiffness is determined as the ratio between the lateral resisting force \( H \) and the lateral displacement (eq. 14). At the occurrence of the first crack the stiffness is calculated as in the eq (15). 

\[
K_c = \frac{H_{cr}}{d_{cr}}
\]

From the correlation curve of the stiffness and deformation the eq. 16 is obtained, 

\[
\frac{K}{K_{cr}} = \alpha \left( \frac{d}{d_{max}} \right)^2
\]

The parameters of stiffness degradation \( \alpha \) and \( \beta \) are related to different test procedure, see Tomaževič 1996. 

Moreover, the same author, as a measure of the energy dissipation capacity, proposes the comparison between cumulative input energy and dissipated hysteretic Energy. 

The first one is defined as the work computed by the actuator to push and pull the wall to maximum displacement amplitudes in one cycle of loading. It is calculated as the sum of areas under the positive and negative part of hysteresis loop. The second as the area of hysteresis loop between two consecutive displacement peaks. 

![Figure 58- a) Evaluation of the input energy in One loading cycle; b) Evaluation of dissipated energy in one loading cycle (Tomaževič 1996)](image)

For masonry, the shear parameters are usually determined also by mean of diagonal compression. 

Since the load is applied with an inclination of 45° on the specimen, the diagonal compression test simulates a pure shear stress state where in which the Mohr circle of the stress state is centered in the origins of \( \sigma-\tau \) axis.
Figure 5.9 d: Diagonal compression test. Loading scheme (on the left) and stress state at the centre of the panel in Mohr’s representation (on the right). (Lagomarsino et al. 2010)

In this case the average shear stress $\tau$ is equal to the principal tensile stress $\sigma_I$ given by (Borri et al. 2003):

$$S_s = \sigma_I = \frac{0.707 \cdot P}{A_n}$$  \hspace{1cm} (17)

Where: $P$ is the diagonal compression load and $A_n$ net area of the panel, calculated as follows:

$$A_n = \left(\frac{W + h}{2}\right)tn$$  \hspace{1cm} (18)

Where:

- $W$ is the width of specimen,
- $h$ is height of specimen,
- $t$ is total thickness of specimen,
- and $n=1$ (% of the cross area of the unit that is solid, expressed as a decimal).

According to this interpretation, in ASTM E519-81 $S_s$ in the eq. 10 is the shear strength considering $P$ as the maximum load applied by the actuator.

The diagonal test on masonry is usual implied to determine shear parameters like cohesion and friction coefficient of mortar joints (Mohr–Coulomb type models, like Mann and Müller’s).

In the case of the rammed earth, assuming the Turšnek Čačovič criterion’s hypothesis of isotropy of the material, the equation 17 expresses the diagonal tensile strength (eq 19):

$$f_{dt} = \sigma_I = \frac{0.707 \cdot P_{max}}{A_n}$$  \hspace{1cm} (19)

In table n the values of diagonal tensile strength are determined only for the diagonal compression test performed with the second set-up, since as seen before those ones, are supposed to be the most reliable.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Load [N]</th>
<th>W [mm]</th>
<th>h [mm]</th>
<th>n [mm]</th>
<th>$A_n$ [mm$^2$]</th>
<th>$f_t$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RE_W3.UNR_D2</td>
<td>25181</td>
<td>550</td>
<td>540</td>
<td>70</td>
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<td>0.47</td>
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<td>550</td>
<td>540</td>
<td>65</td>
<td>35425</td>
<td>0.69</td>
</tr>
</tbody>
</table>

Table 28—Diagonal tensile strength

The external reinforcement increase the diagonal tensile strength.
5. Conclusions

The most remarkable result obtained during the experimental program carried out in this work is the good performance of the scaled wall model RE_W2.EXT. The specimen is reinforced by mean single layers of glass fiber mesh placed on the two faces of the wallet with a clay plaster.

In the diagonal compression test it exhibited higher load capacity, ductility and energy dissipation than the unreinforced sample RE_W.UNR, moreover the diagonal tensile strength is increased of the 32% compared to the unreinforced scaled model.

The failure mode was characterized by great deformations in the transverse direction of the load and few and slight vertical cracks, whereas the unreinforced wall was completely split in the middle after the test.

These results have been achieved in spite of the heavy presence of cracks due to shrinkage present on the clay render, crack that didn't compromised so much the bonding effect between the wall and the mesh. The mesh fiber was still well embedded in the render even if detached due the big deformations reached.

Moreover, the behavior exhibited by both samples, externally reinforced and unreinforced one, in the diagonal test D2, is coherent with the one expected for this test typology, i.e., vertical cracks propagated in the loading direction, towards the corner edges, starting from the middle of the specimen. This can be considered a proof of reliability of the test and a confirmation of the effectiveness of the test set-up where claps at the corners placed vertically were provided.

Regarding the internal reinforce system, it noteworthy the contribution of the glass fiber mesh to the compressive strength that was activated after the failure of the internal unreinforced core of the wall.
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6. Further research developments

Nonetheless the experimental campaign it has been quite useful to highlighted some kind of improvements concerning test set-ups and production procedures that could be undertaken considering further developments of the present research. the consistency of the wet mixture could be improved.

Firstly, in order to improve the optimal moisture content could be determined by mean of a series of tests aiming to the definition of the compaction curves.

Secondly, the adhesion of the fiber mesh could be enhanced implying a mesh with wider spacing step, since the bonding effect is conferred to the penetration of soil particles inside the voids of the mesh, especially if the reinforcement is placed internally inside the wall thickness. In this case in fact inside the soil mixture of the walls is coarser than the one used for the render that showed a good embedding effect.

The render, moreover, could be also be modified in term of material composition. The research of Emiroğlu et al 2005, disclosed that a clay plaster with a ratio 1:1 between clay and sand reduces the appearance of cracks due to shrinkage, in a thin, plastic an clayish layer on soil mixture like the one of the plaster. Inserting some kind of additives could be also a solution to be evaluated.

The restraints implied in the diagonal test set-up avoided the debonding of the reinforced plaster, thus foreseeing the presence of transversal connectors would simulate the action of the clamping restraints. Furthermore, evaluating the bonding strength and the adhesion properties of the reinforced clay plaster, could be significant.

The set-up of the in-plane shear could be rearranged in order to simulate better the boundary conditions.

Once achieved satisfactory and reliable result concerning the in plane shear performance of the scaled walls as designed for the present work, the obtained results could be inversely processed from the scale model to the real scale dimension according to the Buckingham π theorem.

Subsequently, performing the same test on full scale specimen could help to verify the reliability of the methodology based on scaled models tests.

Further research paths could be undertaken namely: the effect of the glass fiber mesh around the opening and in the connection between transversal wall could be inquired.

In conclusion numerical model based on the described test campaign could be performed.
7. REFERENCES


AASHTO T 99-10: Moisture-Density relation using a 2.5Kg hammer. American Association of State Highway and Transportation Officials


ASTM E 2126 – 02a: Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Walls for Buildings

ASTM E 519 – 02 Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages


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