Modelling of the behaviour of TRM-strengthened masonry walls

Gemma Mininno

Master’s Thesis

ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS
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Modelling of the behaviour of TRM-strengthened masonry walls
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ABSTRACT

The un-reinforced masonry structures (URM) represent an important percentage of the building heritage, but they have poor performance in seismic condition. Hence the construction industry has been extremely interested in their repair and rehabilitation.

In the last decades, the innovative materials made of continuous fibres embedded in organic matrices (FRP) have been largely adopted to enhance the seismic performance of the constructions. Initially, these materials were designed to be applied to concrete structures, and only later they were adopted also in masonry structures. Although the advantages introduced by these composite products were undeniable, some drawbacks were also observed, mainly related to the organic matrices (e.g. epoxy).

As a consequence, a new type of composite materials, where the organic substrate is substituted with an inorganic matrix (e.g. cementitious or lime-based mortar), were proposed as a solution. These materials on one hand kept the positive aspects of the FRPs (improvement of the shear-resistance and the deformability without increasing the weight of the structure) and on the other hand overcame their disadvantages (poor behaviour in condition of high temperatures or fire, vulnerability to the external agents, low permeability, low compatibility with the masonry substrate). Consequently, mortar-based composites resulted to be mechanically and physically more compatible with masonry substrates and able to satisfy issues related to conservation principles as reversibility and sustainability.

The innovative composite materials made of continuous fibres embedded in thin mortar layers for externally bonded reinforcement of masonry structures, typically referred to as FRCM (Fibre Reinforced Cementitious mortar) or TRM (Textile Reinforced Mortar), have recently received attention from researchers. Despite the recent interest on the use of these materials, the available information regarding their performance when applied to structures or structural components is still scare. For instance, the effectiveness of TRM systems on the seismic performance of strengthened structures is not clear yet.

Given the aforementioned research context, the present thesis has the aim to investigate the effectiveness of the implementation of TRM composite materials on the in-plane and out-of-plane response of a masonry wall. Therefore, different finite element models to study the in-plane and out-of-plane performance have been created. A comparison of their behaviour in unreinforced and reinforced conditions is carried out by means of non-linear analyses, under the effect of a lateral-monotonic load proportional to the mass (pushover) first and then under the effect of a real accelerogram recorded during the L’Aquila earthquake, occurred the 6th of April 2009. The results are presented and discussed critically.

Keywords: Masonry, TRM, FRCM, Strengthening, FEM modelling, In-plane behaviour, Out-of-plane behaviour, Time history analysis.
RESUMO

As estruturas em alvenaria não reforçada (URM) correspondem a uma importante percentagem do patrimônio edificado existente, no entanto, apresentam uma débil resposta a eventos sísmicos. Assim sendo, a sua reparação e reabilitação é de extrema importância para a indústria da construção. Nas últimas décadas, foram amplamente adotados materiais inovadores à base de polímeros reforçados com fibras (FRP) para o melhoramento da resposta sísmica de edificações. Desenhados inicialmente tendo em vista a sua aplicação em estruturas de betão armado, apenas mais tarde foram também adotados para as estruturas de alvenaria. Apesar das inegáveis vantagens que estes materiais apresentam, foram também observados alguns aspetos negativos, essencialmente referentes às matrizes orgânicas (ex.: epoxy).

Como consequência, foi proposto um novo tipo de materiais compósitos, nos quais a matriz polimérica é substituída por matrizes inorgânicas (ex.: argamassas à base de cal). Estes materiais, por um lado mantem os aspetos positivos dos FRPs (melhoramento da resistência ao corte e da deformabilidade, sem aumento de peso da estrutura), e por outro lado ultrapassaram a sua desvantagem (mau desempenho em condições de altas temperaturas ou fogo, vulnerabilidade aos agentes externos, baixa permeabilidade, baixa compatibilidade com o substrato em alvenaria). Assim sendo, estes novos materiais compósitos demonstraram ser mecânica e fisicamente mais compatíveis com substratos de alvenaria, e capazes de satisfazer questões relativas a princípios de conservação (ex: a reversibilidade e a sustentabilidade das intervenções).

Materiais compósitos inovadores à base de fibras contínuas, embebedas em finas camadas de argamassa para reforço pelo exterior de estruturas de alvenaria, conhecidos por FRCM (argamassas cimentícias reforçadas com fibras), ou TRM (argamassas reforçadas com têxteis), tem sido alvo de atenção por parte de vários investigadores. Apesar do recente interesse no uso destes materiais, a informação disponível referente ao seu desempenho quando aplicados em estruturas ou componentes estruturais é ainda escassa. A título de exemplo, a eficácia dos reforços aplicados em sistema TRM, em face de eventos sísmicos, ainda não é clara.

Atendendo ao estado da arte apresentado acima, a presente tese tem como objetivo investigar numericamente a eficácia de implementação de materiais compósitos à base de TRM, na resposta de estruturas de alvenaria tradicional a ações no plano e para fora do plano. Para tal, foram criados diferentes modelos de elementos finitos para avaliar o desempenho destes materiais para ambos os tipos de ações. Comparou-se também o seu comportamento em situações com e sem reforço, recorrendo a análises não lineares, primeiro sob efeito de cargas monotónicas proporcionais à massa, aplicadas lateralmente, (pushover), e depois sob o efeito de cargas dinâmicas representadas por acelerogramas reais recolhidos durante o sismo de L’Aquila, ocorrido a 6 de Abril de 2009. Os resultados obtidos são apresentados e discutidos em detalhe.

Palavras chave: alvenaria; TRM, modelação FEM, comportamento no plano, comportamento para fora do plano, análise temporal
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RIASSUNTO

Le strutture in muratura non rinforzate (URM) costituiscono una percentuale significativa del patrimonio costruito, ma non presentano delle buone performance in condizioni sismiche. Per questa ragione l'industria delle costruzioni si è dimostrata estremamente interessata agli interventi per la loro riparazione e riabilitazione.

Negli ultimi decenni, questi materiali innovativi costituiti da fibre immerse in matrici organiche (FRP) sono stati ampiamente usati per migliorare il comportamento di queste strutture in condizioni sismiche. Inizialmente, tali materiali erano stati progettati per essere applicati prevalentemente su strutture in cemento e solo negli ultimi tempi il loro utilizzo è stato ampliato anche alla muratura. Nonostante gli indiscutibili miglioramenti introdotti dai materiali compositi, alcuni svantaggi sono emersi, dovuti principalmente alla natura organica delle resine (ad esempio epossidiche).

Di conseguenza, un nuovo tipo di materiali compositi, nei quali la matrice organica è sostituita da una inorganica (ad esempio malta a base di cemento o di calce), è stato proposto come soluzione. Questi materiali, se da un lato assicurano gli stessi risultati degli FRP (aumento della resistenza a taglio e delle capacità deforminge senza incrementare il peso della struttura) dall'altro permettono di superare gli aspetti negativi (vulnerabilità alle alte temperature, al fuoco e agli agenti esterni, bassa permeabilità e ridotta compatibilità con la muratura sottostante). I materiali compositi a base di malta consentono il soddisfacimento dei principi del restauro, quali reversibilità e sostenibilità, poiché risultano più compatibili con il substrato in muratura, sia per quanto riguarda gli aspetti meccanici che fisici.

I nuovi materiali compositi, costituiti da fibre immerse in un sottile strato di malta, per il rinforzo delle strutture in muratura, di solito definiti FRCM (Malta cementizia rinforzata con fibre) o TRM (Malta rinforzata con tessuto), hanno di recente ricevuto molto interesse dal mondo della ricerca. Nonostante il recente interesse nell’uso di questi materiali, le informazioni disponibili riguardo il loro funzionamento quando applicati a strutture reali o a componenti strutturali, sono ancora esigue. Ad esempio, l’efficienza dei sistemi TRM per il miglioramento della risposta, in condizioni sismiche, delle strutture rinforzate non è ancora totalmente chiaro.

La presente tesi, inserendosi nell’ambito di ricerca descritto, ha lo scopo di determinare se l’utilizzo di materiali compositi TRM sia valido o meno quando applicato sulle strutture, in condizioni di sollecitazione nel piano e fuori dal piano. Pertanto, diversi modelli agli elementi finiti sono stati realizzati per studiare questi due tipi di risposta. Un confronto del loro funzionamento in condizioni non rinforzate e rinforzate è stato condotto in termini di analisi non-lineari, sia sotto l’effetto di un carico laterale-monotonico proporzionale alla massa (Push-over) che sotto l’effetto di un carico dinamico rappresentato da un accelerogramma reale, registrato durante il terremoto verificatosi a L’Aquila (6 Aprile 2009). I risultati ottenuti sono presentati e discussi criticamente.

Parole chiave: muratura; TRM, FRCM, rinforzo, modellazione FEM, comportamento nel piano, comportamento fuori dal piano, Analisi Time history.
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Modelling of the behaviour of TRM-strengthened masonry walls
1 INTRODUCTION

1.1 Context and Motivation

The outcome of several seismic events occurred in the last years have pointed out that many of the major failures occur in masonry constructions. The Masonry Society (TMS) and the Federal Management Agency (FEMA) analysis of the damages provoked by several seismic events led to the conclusion that masonry structures fail more often than other type of structures. Moreover, the loss is not only in terms of property but also, and more importantly, in terms of human lives (Babaeidarabad, 2013). These observations together with the fact that the un-reinforced masonry structures (URM) represent an important percentage of the building heritage, have led to extensive effort in development of suitable techniques for repair and rehabilitation of these structures.

Masonry structures, while showing an excellent performance under the effect of gravitational loads, have a very weak response to horizontal loads. The most common effects of an earthquake on a masonry construction are overloading, dynamic vibrations, settlements, in-plane and out-of-plane deformation. More specifically, URM walls are not usually able to withstand in-plane and out-of-plane loading and depending on the material characteristics they may exhibit a brittle failure mode. Clearly, the possibility for this type of failure to occur increases as the quality of the masonry reduces.

In the last decades, large investments have been directed to retrofitting and strengthening masonry structures and enable them to work in safe conditions. New approaches have been searched because the traditional techniques resulted in many occasions time-consuming, expensive and inappropriate for the structural system (for example adding a significant amount of weight to the structure).

Starting from the 1980s, the construction industry for strengthening focused their research on a new composite material: fibre-reinforced polymer (FRP). These systems were studied and developed worldwide and one of the field of application of this new technique was reinforcement of masonry structures. It has been proven that the application of the FRP composites (adopting different composition for the matrices and different material for the fibres) can effectively enhance the shear and flexural behaviour, as well as the ductility of the masonry constructions.

Even though the performance of the FRP materials resulted excellent from several points of view, some drawbacks were also observed, mainly related to the organic substrate: poor fire resistance, poor behaviour at high temperature, lack of vapour permeability and low reversibility. Hence, substitution of the organic binder (e.g. epoxy) with an inorganic binder (e.g. cementitious matrix), which doesn’t exhibit the same disadvantages was proposed. These composites have received extensive attention from the scientific community during the last years and are called with several terms including Fibre-Reinforced-Cementitious Matrix (FRCM) or Textile Reinforced Mortar (TRM).
1.2 Research Gaps

Externally bonded reinforcement of the structures with TRM composites is a relatively new approach when it comes to applications on masonry structures. This is confirmed by the scarceness of studies about this topic in the literature. Only recently, because of the advantages that this technique seems to introduce in the masonry shear behaviour, the interest in this subject has been increasing. Moreover, almost all the case studies investigated so far, mainly deal with the in-plane shear behaviour of the TRM-strengthened masonry.

Therefore, there is a need to study more in depth the effectiveness of these composite materials, both in terms of experimental testing and of numerical analysis.

The research gaps can be divided in two parts:

1) *Experimental tests on strengthened walls.* While the amount of tests carried out on masonry walls are quite numerous, when it comes to experimental works on walls reinforced with composite materials, the number reduces drastically. This condition is even worse with respect to out-of-plane shear tests (Babaeidarabad, 2013). Additional experimental tests which provide a better knowledge on the effectiveness of the reinforcement scheme, the changes in the failure modes, the reinforcement contributions and mechanisms and the behaviour at the interfaces (masonry/TRM and within the TRM between the mortar and the fibres) are required;

2) *Numerical modelling.* The number of numerical simulation of the behaviour of TRM-strengthening are not so many. Among the few studies it is possible to mention (Garofano et al., 2016, Basili et al., 2015, Wang, SAHC 2015). These researches however deal with the study of model characterized by very simple geometrical configurations and they are mostly with regards to the in-plane shear behaviour. In this thesis (Chapter 3), a possible approach for modelling the TRM is presented. Nevertheless, ulterior attempts should be carried out and validated by comparison with adequate experimental results.

1.3 Thesis Outline

The main objective of the thesis is to determine, by means of numerical analyses, the efficiency of the TRM-strengthening technique. To this extent a comparison between the behaviour of an unreinforced masonry wall (URM) and a TRM-reinforced masonry wall (RM) is carried out. Two different models have been investigated in order to assess the shear in-plane and out-of-plane responses.

A macro-modelling strategies has been adopted both for the unreinforced masonry model and for the TRM-strengthen model. With regards to the RM models, the masonry substrate/reinforcement layer is modelled assuming a perfect bond. Moreover, also for the interface mortar matrix/reinforcement grid a no-slip behaviour is considered. The reason behind this simplification is that the aim of this thesis work
is more about understanding how the presence of an TRM-strengthening enhance the shear performance of the masonry wall rather than understanding the specific functioning of the TRM-composite itself. The comparison is performed carrying out first a non linear analysis under a compressive and monotonic-lateral loading (Push-over Analysis) and in a second moment a non linear dynamic analysis (Time History Analysis) where a real accelerogram (L’Aquila Earthquake) was assigned as a base load.

The present thesis is articulated in six chapters. The following chapter (Chapter 2) is a brief review about the literature concerning masonry structures, more specifically the most common failure in-plane and out-of-plane mechanisms. Then a description of the TRM-strengthening technique and finite element modelling strategies are presented. In Chapter 3 the specific finite element strategies, the definition of the models in terms of types of elements and material models adopted, is described. Chapter 4 is dedicated to the outcomes of the analyses carried out in order to investigate the in-plane shear response. The results of the the Push-over Analyses (displacements, principal stresses and strains, load-displacement capacity curve) and of the Time History Analyses (base shear, relative displacements, cracking pattern) for both the unreinforced and the reinforced model are reported. Chapter 5, on the other hand, deals with the study of the shear-out-of-plane response of the masonry wall. Finally, in Chapter 6 a summary of the conclusions yielded from the thesis research are presented and critically discussed. Moreover, recommendations and possible future developments of the topic are proposed.
Modelling of the behaviour of TRM-strengthened masonry walls
2 STATE OF THE ART

2.1 Introduction

Masonry is one of the oldest construction materials and its seismic vulnerability is, by now, a well-known issue. It has been observed, in several occasions, that these structures are vulnerable to seismic actions especially when subjected to both in-plane and out-of-plane loading, even if an earthquake with a low magnitude occurs. Furthermore, their failure is mostly brittle and followed by scattering of debris.

The high variability and complexity of masonry constructions make strengthening interventions a difficult task. The choice of the reinforcement shall take into consideration not only the achievement of a proper safety level but also the compliance with the conservation principles. Moreover, the aim of any reinforcement technique should not be only the increase in terms of stiffness but also the ductility.

In the last years, among the modern techniques, the externally bonded reinforcement with composite materials has been extensively used. The Fibre Reinforced Polymers (FRP) were the first to be proposed, showing a good ability to enhance shear capacity and pseudo-ductility capacity. The rapid diffusion of this method, however, was not supported by a proper experimental evaluation of its effectiveness and compatibility with the substrate. It is found that, especially for the historical constructions, the application of FRP’s brings some drawbacks which encourages researchers for investigating new solutions. Maintaining the advantages related to the composites, substituting the resins matrix with a more compatible inorganic mortars was proposed as a solution. This alternative reinforcement is made of fibre grids embedded in inorganic matrices and is called with several terms in the literature such as Fabric Reinforced Cementitious Matrix (FRCM) or Textile Reinforced Mortar (TRM).

In the following chapter, a review about the in-plane and out-of-plane behaviour of masonry walls is presented. Successively, a description of the state of the art of the TRM technique applied to historic masonry structures is given. Finally, a discussion about the numerical modelling approaches of unreinforced and reinforced masonry is proposed.

2.2 Behaviour of unstrengthened masonry (URM)

Historical masonry constructions are usually designed taking into account gravitational loads, thus the performance of unreinforced masonry walls under these loading conditions is at its best. Major issues arise, instead, when they are subjected to seismic actions.

In fact, in these circumstances they should be able to carry the vertical loads and act as shear walls simultaneously. In heritage constructions, the way in which the masonry responds to a seismic action
is influenced by a number of factors. First of all, the typology of the building in which the wall is located (churches, large town houses also known in Italy as “palazzi”). Secondly, an important role is played by the quality of the materials used, the evolution of their mechanical characteristics during the years, their deterioration and also the effects of damages and interventions performed on the construction (Valluzzi et al., 2014). The potential vulnerability of masonry to seismic actions can be explained by some of its intrinsic characteristics (Wakabayashi, 1986; Allen et al., 2015):

1) The material is brittle (low value of ductility) and when it is subjected to cyclic loads a severe strength deterioration is likely to occur;
2) Its heavy weight lead to large seismic masses and, in turn, to large inertial forces;
3) Masonry structures are characterized by high stiffness and, as a consequence, by short fundamental period. Thus, they are especially vulnerable to earthquakes with a low period because resonance can occur;
4) Frequently, poor detailing can be found;
5) Masonry properties may differ largely from one country to another. Therefore, the strength may vary widely. In general, however, the values of the tensile and the shear strength are low.

Usually, in historical constructions two main structural components can be found (Magenes et al., 1997), (ElGawady et al., 2005):

1) Load-bearing walls organized in orthogonal planes;
2) Floor diaphragms, that usually are not rigid enough to constrain the out-of-plane movements of the walls.

The observation of damages in the afterward of several seismic events has allowed to identify two principal categories: in-plane and out-of-plane failures. Consequently, a full assessment of the behaviour of URM buildings should take into account both of them. There are different conditions that can cause failure. In general, the in-plane behaviour of a wall is completely related to the characteristics of the wall itself. Materials, geometry, presence of doors or windows are some of the factors that can influence their capacity. Conversely, the out-of-plane mechanisms are provoked by large deformation of the floor structures or by the absence of efficient restraints.

2.2.1 In-plane behaviour

In the hypothesis that the out-of-plane behaviour is restrained thanks to correct design expedients, the in-plane walls have the role to guarantee the stability of the structures. Usually these walls are characterized by the presence of openings (windows and doors), which effectively reduce the resisting
section of the wall. As a consequence, the remaining piers have the role to carry the vertical and the lateral loads.

The in-plane behaviour of unreinforced masonry walls depends on a number factors. Primarily from the geometry of the piers, their boundary conditions and the magnitude of the horizontal force. Then, the components of the masonry (blocks, mortar and block/mortar interfaces) play a fundamental role.

The flexural response of a wall is governed by the rocking of the piers more than by the beam-type behaviour, while the failure mode is dominated by the shear, producing either a diagonal cracking or a sliding on the planes of the mortar joints. Moreover, the response of masonry walls is strongly influenced by the loading rates, the boundary conditions and the level of pre-compression (the value of the axial force). Another crucial aspect about this composite material is its non-homogeneous nature, which makes its behaviour not predictable and variable from case to case.

Figure 1 - In-plane failure modes of a laterally loaded URM (ElGawady et al., 2005): a) Shear cracking; b) Sliding Failure; c) Rocking Failure; d) Toe crushing.

Nevertheless, the typical failure modes under in-plane actions are as follows (Figure 1):

1) **Shear cracking** (Figure 1-a). The peak resistance is governed by the opening of a diagonal crack which may follow the stepped path of the head and the bed-joints or break the blocks, depending on the relative strength of the mortar, the stones (or bricks) and the interfaces. Usually, this kind of failure occurs when the aspect ratio (height/length) is small or the axial force is large.

2) **Sliding Failure** (Figure 1-b). This type of mechanism can develop in the case of overcoming of the mortar tensile strength in the bed-joints. Then, because of the reversed seismic action horizontal sliding planes can appear here. The favourable conditions for this mechanism to activate are a low axial force and/or a low friction coefficient.

3) **Rocking Failure**. When the horizontal displacement or force increase and exceed the tensile capacity of the bed-joints, the shear load is carried by the compressed masonry. This can result in overturning of the wall at the heel (Figure 1-c) and sometimes simultaneously by the crushing in compression of the toe (Figure 1-d).
2.2.2 Out-of-plane behaviour

Recent earthquake events have demonstrated that out-of-plane failures are the principal motives of the seismic vulnerability of masonry buildings. The main reason is that, usually, a wall can withstand larger horizontal forces when they act in its plane. This, together with the low level of tensile strength of the material, can lead to a high probability for the walls to fail by overturning or by bending. There are several aspects that can influence the activation of an out-of-plane mechanism (Benedetti et al., 2011):

1) *Height of the walls.* The higher is the wall the more instable it will be.

2) *Connections with the orthogonal walls.* Basically, the level of connection at the corners with the transversal walls determine the way in which the wall fails. These walls provide the restraint against the overturning by means of friction between the blocks on the contact surface. Therefore, the overlapping of the units at the corners is a key factor. The absence of these connections provoke the decomposition of the building in a number of macro-elements, each of them behaving separately from the others. As a consequence, in an existing masonry building a local failure mechanism can lead to failure before a global collapse mechanism develop.

3) *Roof and floor configuration.* The stiffness and the orientation of the horizontal diaphragms play an important role too. More specifically, from them depends the possibility of the wall to withstand local flexure.

With regards to this topic, ReL UIS, a network of seismic engineering experts elaborated, after an accurate study of the damages caused by recent and past earthquakes, a set of guidelines that inexperienced engineers could follow in order to identify the collapse mechanisms. Among all the possible local failure mechanisms for masonry buildings, the most dangerous and consistent is the
turning of masonry walls around the base edge, or the formation of an internal cylindrical hinge in the panel (“Ribaltamento semplice”, Figure 2). There are some variations of this same mechanism, according to the portion of the wall interested by the movement:

1) One or more levels of the wall, depending on the restrain given by the floor;

2) One or more leaves composing the wall, depending on the masonry characteristics and layout;

3) Different geometry of the wall, depending on the presence of openings (doors and windows).

The main reasons which lead to this type of movement are the absence of connection with the transversal walls and with the roof at the top. These conditions may be due to poor detailing or to some deficit in the design of the structure: flexible floor diaphragms, bad interlocking at the intersections of the wall, presence of unrestrained pushing structures, multi leaves with no transversal interlocking or low quality masonry walls. In these conditions, the activation of the mechanism is usually evidenced by some typical symptoms:

1) Vertical cracks extending along the intersection with the transversal walls (Figure 2).

2) Out-of-plane of the façade interested by the overturning;

3) Pull-out of the floor beams.

It is not rare to find an out-of-plane mechanism which involve not only the interested wall but also some portions of the transversal walls (“Ribaltamento composto di parete”, Figure 3).

![Figure 3 – overturning mechanism of a wall, “Ribaltamento composto di parete” (ReLUIS, 2008) from left to right: Observed damage; Schematic representation (hinge at the base); Schematic representation (internal hinge).](image)

As in the previous case there may be some different variant of the same mechanism, namely the following:

1) One or more levels of the wall, depending on the restrain given by the floor;
2) The geometry of the macro-element can vary depending on some characteristics of the orthogonal wall (quality of the masonry material, presence of openings, different typologies of diaphragms at each level). More specifically, what will change is the shape of the wedge of the transversal wall.

Unlike the “ribaltamento semplice” this mechanism develops with higher probability when the connection with the roof above is missing, but a good interlocking with the transversal wall can be found. Usually the absence of rigid diaphragms, the non-box behaviour, the presence of openings on the transversal walls and the low strength properties of the masonry are all factors that could lead to failure according to this local mechanism.

Moreover, when analysing the damage survey some recurring features may be listed:

1) Diagonal cracking on the orthogonal walls;
2) Out-of-plane overturning of the façade;
3) Pull-out of the floor beams.

Sometimes, when there are some concentrated forces on the top corner of two walls intersecting (for example the roof beam imposing a horizontal force) another well-known mechanism can occur, specifically the overturning of a corner wedge (“Ribaltamento del Cantonale”, Figure 4)

Figure 4 – Overturning mechanism of a wall, “Ribaltamento del cantonale” (ReLUIS, 2008) from left to right: Observed damage; Schematic representation.

Another important category of mechanisms is represented by those involving not an overturning movement but bending: vertical and horizontal, depending on the direction of the cylindrical hinge.

The vertical bending is described by the static scheme represented in Figure 5: division of the façade in two blocks which rotate about an hinge developing in the horizontal direction. This specific situation is more likely to happen when there is a good connection between the wall and the horizontal
structures on top, while there is a poor interlocking with the transversal walls. There are several factors which are able to govern the activation of the flexural behaviour of the wall, like the slenderness of the element, the poor connection with the intermediate diaphragms or the absence of a good interlocking among the different leaves composing the wall. In each specific situation (different boundary conditions and materials), the mechanism can have diverse features.

Figure 5 – Vertical bending mechanism of a wall, “Flessione Verticale” (ReLUIS, 2008) from left to right: Static scheme; Schematic representation (single floor); Schematic representation (multi-storeys).

It could involve the whole façade or part of it, depending on the level of restraint given by the intermediate horizontal structures (Figure 5). It could also regard one or more leaves composing the wall, varying the level of transversal interlocking in the section of the wall. The geometrical features of the mechanism depend on the presence and on the level of the thrusting force induced by structural elements (arches and vaults). With the aim to identify the mechanism some exterior signs on the wall can be observed: bulging and out-of-plumb of the wall, horizontal and vertical cracks, pull-out of the floor beams.

One last mechanism from which a masonry wall can be characterized is the Horizontal Bending or “Flessione Orizzontale” (Figure 6).

Figure 6 – Horizontal bending mechanism of a wall, “Flessione Orizzontale” (ReLUIS, 2008) from left to right: Observed damage; Schematic representation.
In this circumstance masonry material is expelled from the upper part of the wall. Furthermore, wedge-shaped portions of the facade rotate about cylindrical hinges (vertical and with a 45° axis). Thus, the failure could happen or for horizontal instability (for isolated constructions) or for crisis of the material (for buildings in aggregates). Unlike the previous bending mechanism, this one has higher probability to occur when there is no connection between the wall and the roof, while the interlocking with the orthogonal wall is good. It could manifest in different modalities: affecting one or more levels of the building, a portion or the whole thickness of the wall.

### 2.3 Strengthening of URM walls with TRM composite technique

In order to overcome the deficit of the unreinforced masonry walls, in terms of seismic performance both in-plane and out-of-plane, a variety of traditional and innovative strengthening techniques can be found in the literature and in the common practice.

More specifically, this thesis focuses on an alternative method to the FRP externally bonded reinforcement, that has been proposed and studied in these last years: Fibre Reinforced Cementitious Matrix (FRCM) or Textile Reinforced Mortar (TRM).

The TRM technique keeps the advantages of the Fibre Reinforced Polymers method and overcome its principal drawbacks, especially in the field of the historical structures. Both methods adopt fabrics or grids embedded in a matrix, but the different composition of this adhesive support is what distinguishes them: the cement-based matrix substitutes in TRM the FRP's epoxy resin. This substitution has the purpose of avoiding the negative outcomes observed in the application of the FRP, namely (Valluzzi et al., 2014) brittle failure, the sensitivity to impact, notching and environmental agents, the incompatibility with the masonry substrate and difficult removal and the poor performance at high temperatures.

Despite the increasing interest and diffusion of the TRM composites, there are some aspects that still need to be examined in depth, such as:

1) A clarification on the critical aspects related to application technologies (bonding and anchorage of the textiles);

2) Standardization of the methods and the experimental procedures, in order to define proper design and assessment criteria;

3) An evaluation of the effectiveness and the duration of the interventions;

4) The required criteria to consider the application of this technique in the field of historical constructions.

Furthermore, the effect of this alternative method on the structural behaviour and failure modes should be investigated. As a matter of fact, implementation of TRMs has an effect on the ultimate strength of the masonry, but its consequences on the displacement capacity is not completely clear.
2.3.1 Characterization of the materials

TRM consists of a sequence of one or more layers of fibre reinforcement (mesh or grids) applied to the substrate surface by means of a mortar matrix.

The matrix function is to protect and wrap the fibres and to transfer the stress from the support (concrete or masonry) to the reinforcement. The performance of the TRM strengthening technique is strongly governed by the composition of the cementitious matrix. It may be made of hydraulic or non-hydraulic mortar, but in any case it should be non-shrinkable and easily workable. Usually, it is made of Portland cement with the addition of dry organic polymers in a low dosage (less than 5% in weight), whose presence is needed to ensure proper workability of the fresh mix, to control the hardening rate, to improve the bond with the fibres and to enhance the mechanical properties (Nanni, 2012). The type of support on which the TRM will be applied may also be a factor in the choice of the type of matrix: the cement-based mortar is usually used for existing structures, while the lime-mortar is normally more suitable, in terms of compatibility with the substrate, for historical masonry structures.

The function of the reinforcement is to carry the tensile stress. In TRM, the fabric sheets usually used in FRPs are substituted by open meshes (the spacing is not larger than 2 cm) in which the yarns are organised in a two or more directions layout (preferably orthogonal) by means of weaving, tufting, knitting or braiding. The "open" fabrics is essential for this type of reinforcement because of the larger dimensions of the grains characterizing the mortar, which prevent the matrix from penetrating and impregnating the fibres. This arrangement allows, however, to obtain a wider matrix-reinforcement interface. The fabric grids are usually made of carbon, aramid, basalt, alkali-resistant glass, PBO (polyparaphenylene benzobisoxazole) or hybrid systems (Figure 7).

![Continuous reinforcement: steel, carbon, PBO](image)

The application of the reinforcement is also an important aspect. Firstly, the support surface should be prepared (Mantegazza, 2006): cleaned (by sandblasting, grinding or other similar abrasive techniques) to remove refinement materials or weak surfaces and wetted with water to avoid shrinkage phenomena on the cementitious matrix. Then a thin layer of mortar is applied on the masonry surface and the fibre grid or mesh is pressed on it. A light pressure shall be applied in the direction of the fibres. These operations can be repeated as many times as needed to reach the desired value of
strength. The last step is the application of a final layer of mortar, whose function is protection and embedding of the fibre net and the refining of the support surface.

2.3.2 Mechanical behaviour

The behaviour of a TRM strengthening system is highly influenced by the ultimate tensile strength of the textile and the substrate-to-reinforcement shear bond performance, which in turn depends on the characteristics of the substrate, the matrix and the textile-to-matrix interaction. While the tensile strength is crucial in some applications in which the textile failure may occur first, such as in the confinement of columns/pillars or the extrados reinforcement of arches and vaults, the shear bond behaviour is fundamental in a wider range of circumstances where the failure at the interface between the substrate and the reinforced mortar is more likely to occur, for example in strengthened walls against in-plane and out-of-plane loadings. In fact, the reinforcement remains effective as long as the mortar is able to transfer the stress from the structure to the textile, the component of the composite material responsible for carrying the loads.

Several direct tensile tests (Ascione et al., 2015) have shown that TRM systems response under a tensile loading is characterized by three stages (Figure 8): un-cracked (I), crack development (II), cracked (III).

![Figure 8](image)

**Figure 8 –** Typical stress-strain response curve of TRM systems under tensile loading: un-cracked, crack development, cracked stages (Ascione et al., 2015).

During the first stage the reinforcement layer is undamaged, thus its response is linear. Then, the beginning of stage II is identified by the appearance of the first crack. From this moment on the increasing number of cracks cause the reduction in stiffness. In these first two phases the behaviour of
the composite material is influenced by the mechanical characteristics of the mortar, the textile and also of the textile-mortar interface, from which the stress transfer depends. These parameters determine the crack width and distribution and in turn the durability of the reinforcement. At some point the crack pattern stabilize and a slight increase in stiffness is registered. This the point of transition to the third phase: an increase in strains produces a widening of the cracks and the load bearing capacity and the elastic modulus are related to the textile component. In this stage the mortar matrix may still be able to carry some loads, but it has mainly a redistribution of the transversal loads function. The failure of the system in the majority of the cases happens because of the textile rupture for tension: after the first cord breaks the damage will propagate in all the others.

Concerning the shear bond behaviour different failure modes can develop according to the shear strength of the mortar matrix, the tensile strength of the textile and the textile-to-mortar bond/interlocking (Figure 9): (A) debonding with cohesive failure in the substrate; (B) failure at the reinforcement-to-substrate interface; (C) failure at the textile-to-matrix interface; (D) sliding of the textile within the reinforcement thickness; (E) tensile rupture of the textile in the unbonded portion; (F) tensile rupture of the textile within the mortar matrix.

Figure 9 – Failure modes in shear bond tests on externally bonded TRM strengthening systems: (A) debonding with cohesive failure in the substrate; (B) failure at the reinforcement-to-substrate interface; (C) failure at the textile-to-matrix interface; (D) sliding of the textile within the reinforcement thickness; (E) tensile rupture of the textile in the unbonded portion; (F) tensile rupture of the textile within the mortar matrix (Ascione et al.,2015).

Each of these failure modes correspond to a different shape in the force-slip curve (Figure 10). In the first three modes after a slip curve an almost flat branch is visible, followed by a brittle failure (a). In Failure Mode D, on the other hand, there is a soft load decrease due to the loss of friction of the textile sliding inside the mortar (b). In Mode E a tensile failure of the textile can occur before the flat branch of the curve is reached, resulting in an instantaneous reduction of the load (c). Finally, Mode F is characterized by a sudden decrease of the force followed by a further slower decrease of the load. This last behaviour is due to the telescopic rupture of the wires.
Modelling of the behaviour of TRM-strengthened masonry walls

Figure 10 – Typical force-slip curves observed in shear-bond tests on externally bonded TRM strengthening systems related to: (a) failure modes A, B, C; (b) failure mode D; (c) failure mode E; (d) failure mode F.

2.3.3 Advantages and disadvantages

Studies on this new material (Papanicolaou et al, 2007) have shown that textile-reinforced mortar systems allow to increase the load carrying capacity of URM of about 30%. If on one hand this method shows a lower effectiveness compared to the traditional FRP’s, on the other hand it consents to increase the deformability up to 15-30%. In general, however, the increase in strength depends on the number of layers and the axial load, but it is important to keep in mind that a larger number of layers means a decrease in ductility (Garofano et al, 2016).

There are several reasons because of which the FCRM technique results more suitable for the historical constructions:

1) the relatively low cost of manufacturing and time of installation;
2) the reduced thickness of the mortar layer, which doesn’t increase the mass of the strengthened wall;
3) the higher compatibility with the old masonry substrate (clay bricks, tuff, stone blocks);
4) the higher vapour permeability;
5) the durability to external agents;
6) the higher reversibility.

All these advantages are added to the high tensile strength, the high stiffness-to-weight ratio and the corrosion and fatigue resistance, which already characterized the FRP composites.

On the other hand, TRM have some drawbacks, such as:

1) lower quality of adhesion between the fabric and the matrix;
2) brittleness of the matrix.

This latter characteristic is crucial. In fact, the mortar-fabric interface is extremely important as it governs the behaviour of the system reinforcement-structure. It is responsible for the stress
transferred between the filaments within a yarn and between the yarn and the matrix. The inorganic matrix doesn’t have the same adhesive properties of the epoxy, thus the matrix-fibre bond is not as strong as in the FRP’s. Furthermore, the cement-based mortar is not able to fully penetrate between the filaments that constitute the fibres because of the too large dimension of its grains. As a consequence, the inner and outer filaments bond with the matrix are different and this lead to a type of failure called “telescopic”: a progressive layer-by-layer break down process that starts at the sleeve and end at the core filaments.

2.3.4 The use of TRM strengthening in historical buildings

Reinforcement with composite materials for what it may concerns masonry in historic buildings could be proposed especially in the following cases (Valluzzi et al., 2014):

1) Overall or partial overturning of façades or corners of the buildings. In order to counteract the movement and to improve the connections among the structural elements.

2) In-plane and out of plane strengthening. In order to increase the load-bearing capacity and the stiffness of the walls under shear and bending loads.

3) Confinement under vertical loads. In order to improve both the strength and the ductility of columns and piers.

4) Bonding support for curved shapes. In order to enhance the load-bearing capacity of vaults and arches and also to reduce their horizontal thrust.

5) Presence of cracks. In order to repair or limit the opening of the cracks.

2.4 Finite Element Modelling

Nowadays the finite element method is a very powerful tool to understand the structural behaviour of components. However, in order to obtain an accurate description of the reality, the mathematical formulation should be matched with the experimental evidences. The objective of the combination of laboratory simulations and FEM analysis is, among other things, the definition of a constitutive model (Lourenço, 1998), that is the description of the relation between the stress and the strain tensors in a point of a solid body.

It should be mentioned that a model will not describe fully all the mechanisms of a material, but it is more a simplified representation of its true behaviour. The goal of this innovative method is to provide a robust numerical tool, able to describe efficiently the different stages of the performance of the material: linear elastic, cracking and degradation, complete loss of strength (failure).

The description of the functioning of masonry structures (reinforced and unreinforced) through FEM is more difficult because of the complexity of this composite material. This is even more true in the case of historical buildings where the regularity in geometry is missing, the constitution of the inner core of
the structural elements is unknown, a complete mechanical characterization is hardly possible and the information about the historical evolution is not easy to obtain (Lourenço, 1998).

Only recently, the interest in developing sophisticated tools to describe the masonry behaviour has been increasing. In this paragraph a brief description of the possible approaches to model the masonry and the TRM reinforced masonry are presented.

2.4.1 Modelling masonry

Masonry is a material characterized by a wide heterogeneity, but in general it is made of two principal elements: units and joints. There can be numerous combinations of different materials: bricks, blocks, ashlars, adobes, stones for the units and clay, bitumen, chalk, lime, cement based mortars for the joints. However, a common feature that characterize the mechanical behaviour of all the types of masonry is the very low tensile strength.

Masonry exhibits an anisotropic behaviour due to the presence of the mortar joints, which represent planes of weakness. Depending on the aim of the analysis, there are several modelling strategies for this composite material characterized by different levels of accuracy and simplicity. Namely, they could be summarized as follow:

![Diagram of masonry composite material](image)

**Figure 11** – Modelling strategies for masonry composite material: a) Detailed micro-modelling; b) Simplified micro-modelling; c) Macro-modelling.

a) *Detailed micro-modelling* (Figure 11-a). The blocks and the mortar are modelled separately adopting continuum elements. On the other hand, the unit-mortar interface, representing a potential crack/slip plane, is designed by means of discontinuous elements. In this method, Young’s modulus, Poisson ratio and eventually some others inelastic properties are taken into account.

b) *Simplified micro-modelling* (Figure 11-b). Differently from the previous strategy, the joints and the unit-mortar are discretized together lumping their properties in only one discontinuous element. The units, instead, are still modelled using continuum elements, but their dimensions are increased in order to maintain the geometry unchanged. As a consequence, the material
is considered as an assembly of elastic blocks joined by potential fracture/slip lines, whose Poisson’s effect is lost since this property is not defined for the mortar.

c) **Macro-modelling** (Figure 11-c). This approach is different compared to the previous ones. In fact, the mechanical characteristics of the components of the masonry material (units, joints, unit-mortar interfaces) are homogenized and modelled adopting anisotropic continuum elements.

The choice of the strategy to be adopted is usually related to the field of application. More specifically, the micro-modelling approaches are implemented in models of structural details or when the study of the local behaviour of the masonry structures is required. On the contrary, the macro-modelling is used in those cases in which the object of the analysis is the overall performance of the structure. It is clear that this strategy is less time consuming and less computationally demanding, thus it is used when a compromise between efficiency and accuracy is needed.

### 2.4.2 Modelling TRM strengthened masonry

The level of complexity which characterize the modelling of reinforced masonry is most certainly higher than for URM because of the number of components involved in the system: the masonry itself, the type of reinforcement and the interface between the reinforcement and the substrate.

In the literature it is possible to find different approaches to represent the strengthening of masonry structures by means of composite systems. In general, however, they can be divided in two groups:

1) **Micro-modelling.** Models in which each component (the masonry, the reinforcement, the mortar and the interfaces) is modelled separately. More specifically, non-linear interface elements are adopted to reproduce the behaviour at the substructure-mortar and mortar-reinforcement boundaries.

2) **Macro-modelling.** Models in which the reinforcement behaviour is included in the strengthening support (matrix) or in the masonry elements by means of homogenization;

Clearly, also in this case, the choice of the model depends on the purpose of the study. A micro-model is used to understand the way each element works and how they interact between them. A great interest, for example, has been shown for the bonding aspects. In this case, characterization of the materials need to be matched with analytical and/or experimental studies. When adopting a micro-modelling approach, the definition of all the failure mechanisms for TRM needs to be included: crack propagation in the matrix, the local failure of the textiles (within the yarns or the roving), the bond-slip mechanism between the reinforcement layer and the matrix (Holler et al., 2004).

Adversely, the macro-models are usually adopted when the object of the analysis are simple structural models, such as masonry panels and arches. Usually, the aim in these kind of works is to verify the effectiveness of the reinforcement system in comparison with the unreinforced models. In this case a smeared crack model is adopted, meaning that locally originated cracks are supposed to be scattered
over a wide surface and a tension softening model is associated to the mortar in order to describe the effect of the fibre bond on its tensile cracking. The reinforcement is, most of the time, implemented in the matrix model, adopting an equivalent grid or a bar/truss elements depending on the choice to represent a perfect bond or a slip-bond behaviour at the interface.

The way in which the composite structures behave at the boundaries between the matrix and the textile reinforcement is a crucial aspect when it comes to FEM modelling. Several experimental studies (D’Ambrisi et al., 2012), however, have shown that the structural behaviour of a reinforced wall is controlled by the matrix-substrate interface, because the matrix-grid interface is stronger.

In Wang (2015) an investigation about the the FE modelling of TRM composite materials at a can be found. In this study, two different strategies to model the tensile and the bond behavior of TRM are proposed. In one model the bond between the matrix and the fibres is assumed to be perfect, while in the other a bond-slip behaviour at the matrix/fibres interface is considered. In both cases the mortar is modelled using 8-node curved shell elements, while for the reinforcement two different approach are adopted:

1) To model a perfect-bond behaviour an embedded grid-reinforcement is proposed;
2) To model the slip-bond behaviour the reinforcement is modelled as embedded bar elements. Moreover, an interface between the mortar and the bars is created, to which a bond-slip law is assigned.

With regards to the mortar material a tensile total strain crack model is adopted, and different tensile softening laws based on the fracture energy are compared (exponential and JSCE). For the fibres, instead a linear elastic until failure behaviour is implemented. The FE simulations are then compared with the experimental results of previous researches (Carrozzi et al., 2015; D’Ambrisi et al., 2014).

With regards to the macro modelling approach, in the literature very few study about the finite element simulation of the TRM-reinforcement can be found. Among them it is possible to mention the work done by (Basili et al., 2015), in which it is presented a simplified modelling approach for the analysis of the in-plane shear behavior of tuff stone masonry panels strengthened with a Basalt Textile Reinforced Mortar (BTRM) system. In this study the non-linear behavior of the unreinforced and reinforced panels is reproduced by means of a macroscopic smeared crack approach. The masonry is modelled as an isotropic continuum material characterized by different nonlinear softening laws in tension and compression. The BTRM composite is modelled with two layers representing the basalt textile and the mortar matrix. The numerical results of this model are then compared with experimental output.
3. FEM MODELLING OF THE URM AND TRM-STRENGTHENED WALLS

3.1 Introduction

In this chapter, the modelling strategies for the unreinforced and reinforced walls is described deeply. The choice of the type of mesh, the mechanical constitutive laws for the masonry, the mortar and the reinforcement are presented. Moreover, the interface behaviour between the reinforced mortar and the masonry wall is described.

In this thesis the finite element analysis is adopted in order to understand the different structural responses of a masonry wall, in unstrenthened and strengthened conditions, when subjected to a seismic action. As the aim of the thesis is to study the effectiveness of the reinforcement and the evaluation of the failure mechanisms a macro-modelling approach is adopted. Consequently, the strategy was to implement the reinforcement in the matrix as an equivalent grid, assuming a perfect bond at the boundaries between the mortar and the fibres. Moreover, the behaviour of the mortar-substrate interface is assumed to be perfect as it is supposed to be in real conditions. The FE package DIANA 9.6 by TNO DIANA is used here for simulation and performing the analyses.

3.2 Modelling strategy for the masonry

The behaviour of masonry composite (units and joints) is lumped in continuum elements. Quadrilateral 8-node shells, denoted as CQ40S in the DIANA library (Figure 12) are used for masonry elements.

![CQ40S Elements: 8 node, quadrilateral shell.](image)

These elements are based on an isoparametric approach degenerated from a three dimensional formulation by introducing two shells hypotheses (Diana-9.6 User’s Manual):

1) **Straight-normals.** It assumes that the normal remains straight but not necessarily orthogonal to the reference surface. Transverse shear deformations are included according to the Mindlin-Reissner theory.

2) **Zero-normal-stress.** It assumes that the normal stress in the normal direction of a lamina basis is forced to zero.
The in-plane lamina strains vary linearly in the thickness direction, while the shear strains are forced to be constant. As the effective distribution of the shear strains and stresses in the thickness is parabolic, the constant shearing strains have an equivalent value obtained applying a shear correction factor according to the assumption that a constant shear stress yields approximately the same shear strain energy as the actual shearing stress.

Five degrees of freedom are defined in each node: three translations and two rotations. Furthermore, curved shells should be thin, meaning that the thickness should be smaller than the in-plane dimensions. Force loads may act in any in-plane and out-of-plane directions, while moment loads, should act around axis in the element face (Figure 12).

For curved shell elements, DIANA gives the option to choose different shapes, and according to them it determines the direction in which the thickness is measured. In the models used in this study, the shape of the curved shell was selected to be flat as the typology of the structures analysed belongs to the wall category.

[Diagram of Hill and Rankine type criteria]

Figure 13 – Composite Rankine-Hill yield criterion (Lourenço, 1996).

To simulate the behaviour of the masonry in the finite element model a hypothesis of quasi brittle material is made, taking into account also the cracking phenomenon. A yield criterion based on the plane stress anisotropic yield law developed by Lourenço (1996) enhanced with two new stress components is adopted. The application of a plane stress criterion to shell elements is possible because in both cases the behaviour is two-dimensional. The aforementioned criterion considers a Hill type criterion in compression and a Rankine type criterion in tension. Such a model is able to describe the biaxial stress state of the masonry in which tensile and compressive stresses arise in different principal directions in the same point. In tension an exponential softening law is assumed, while in compression a parabolic hardening followed by a parabolic/hardening law is implemented.

With regard to the masonry model implemented in the case studies object of this thesis the properties are reported in the following table (Table 1).
Table 1 - Masonry mechanical parameters (Lourenço et al., 1995)

<table>
<thead>
<tr>
<th>Masonry Wall Mechanical Parameters</th>
<th>RANKINE-HILL MODEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\nu$ Poisson’s ratio</td>
<td>0,10</td>
</tr>
<tr>
<td>$E$ Young’s Modulus</td>
<td>3960 MPa</td>
</tr>
<tr>
<td>$\rho$ Density</td>
<td>1,90E-09 T/mm$^3$</td>
</tr>
<tr>
<td>$G$ Shear Modulus</td>
<td>367 MPa</td>
</tr>
<tr>
<td>$\sigma_{tx}$ Tensile strength along $x$-axis</td>
<td>0,35 MPa</td>
</tr>
<tr>
<td>$\sigma_{ty}$ Tensile strength along $y$-axis</td>
<td>0,25 MPa</td>
</tr>
<tr>
<td>$\sigma_{cx}$ Compressive strength along $x$-axis</td>
<td>10 MPa</td>
</tr>
<tr>
<td>$\sigma_{cy}$ Compressive strength along $y$-axis</td>
<td>8 MPa</td>
</tr>
<tr>
<td>$G_{fcx}$ Fracture Energy in compression along $x$-axis</td>
<td>20 N mm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fcy}$ Fracture Energy in compression along $y$-axis</td>
<td>15 N mm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fx}$ Fracture Energy in tension along $x$-axis</td>
<td>0,05 N mm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fy}$ Fracture Energy in tension along $y$-axis</td>
<td>0,0018 N mm/mm$^2$</td>
</tr>
<tr>
<td>$\alpha$ Factor that determines the shear stress contribution to the tensile failure</td>
<td>1,00</td>
</tr>
<tr>
<td>$\beta$ Factor which couples the normal compressive stresses</td>
<td>-1,00</td>
</tr>
<tr>
<td>$\gamma$ Factor which controls shear stress contribution to compressive failure</td>
<td>2,5</td>
</tr>
<tr>
<td>$K_p$ Factor that specifies the equivalent plastic strain corresponding to the peak compressive stress</td>
<td>0,0012</td>
</tr>
</tbody>
</table>

3.3 Modelling strategy for TRM composite

With the purpose of a macro-modelling approach the TRM strengthening layer is simulated implementing an additional set of 8-nodes curved shell elements (CQ40S) for the matrix in which the reinforcement is embedded as an equivalent grid (Figure 14). As a consequence, no slip is allowed between the fibres and the mortar.

![Figure 14 – Grid embedded reinforcement in curved shell elements.](image-url)
Moreover, a total strain crack model has been assigned to the matrix, implementing for the tensile behaviour a softening law based on the fracture energy approach. More specifically in this thesis the nonlinear softening with plateau function elaborated by the Japan Society of Civil Engineer (JSCE) has been chosen. On the other hand, the behaviour in compression is simulated by a parabolic function. For the tensile behaviour of the fibres a brittle model with a linear elastic behaviour until failure is used (Figure 15). Specifically, the properties assigned to the mortar are described in Table 2.

![Image] Figure 15 – Non linear material for the TRM composite. From left to right: JSCE uniaxial tension model, uniaxial compressive parabolic model, brittle model for TRM textile.

<table>
<thead>
<tr>
<th>Mortar Mechanical Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TOTAL STRAIN BASED CRACK MODEL</strong></td>
</tr>
<tr>
<td><strong>ν</strong> Poisson’s ratio</td>
</tr>
<tr>
<td><strong>E</strong> Young’s Modulus</td>
</tr>
<tr>
<td><strong>ρ</strong> Density</td>
</tr>
</tbody>
</table>

| **Total strain based crack model**                              |
| **Crack Orientation** Rotating                                   |
| **Crack bandwidth or effective length**                         | 100 mm |

**Tensile Curve: JSCE Tension stiffening**

| **Tensile Strength** | 3,325 MPa |
| **Plateau and strain** | 0,0016 |
| **Power c**           | 0,6 |
| **Reduction due to lateral cracking**                           | No |
| **Poisson’s ratio reduction**                                   | No |

**Compressive behaviour: Parabolic**

| **Compressive Strength** | 15 MPa |
| **Compressive Fracture Energy**                                  | 20,64 N mm |
| **Residual Compressive Strength**                                | |

Erasmus Mundus Programme
Table 3 - Reinforcement mechanical parameters

<table>
<thead>
<tr>
<th>Material</th>
<th>Poisson’s ratio (v)</th>
<th>Young’s Modulus (E)</th>
<th>Tensile strength (f_t)</th>
<th>Diameter (d)</th>
<th>Area of fibres per unit of length (ρ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PBO</td>
<td>0,3</td>
<td>188000 MPa</td>
<td>3400 MPa</td>
<td>0,41 mm²</td>
<td>38,1 mm²/m</td>
</tr>
<tr>
<td>Basalt</td>
<td>0,3</td>
<td>89000 MPa</td>
<td>1542 MPa</td>
<td>0,23 mm²</td>
<td>35,0 mm²/m</td>
</tr>
<tr>
<td>Glass</td>
<td>0,3</td>
<td>72000 MPa</td>
<td>1290 MPa</td>
<td>0,85 mm²</td>
<td>32,3 mm²/m</td>
</tr>
</tbody>
</table>

Figure 16 – Brittle elastic uniaxial model for different types of reinforcement.
Concerning the textile reinforcement, different types (PBO, Basalt, Glass) have been investigated in order to understand which one of them is more efficient in terms of increase of the load capacity of the wall and also in terms of ductility. The characteristics of each strengthening system adopted are showed in Table 3 and in Figure 16.

Furthermore, the boundaries among the masonry substrate and the cementitious mortar matrix a structural interface is created adopting the DIANA plane quadrilateral 8+8 nodes element (CQ48I) which has a three-dimensional configuration (Figure 17).

![Figure 17 – CQ48I Elements: plane quadrilateral, 8+8 nodes, 3-D.](image)

The function of these additional interface elements is to couple the displacements of the masonry wall with the reinforced mortar. It was decided not to allow any relative displacement along the interfaces by adopting a high value of stiffness (1.0 $E+07$ N/mm$^3$) both in the normal and in the tangential directions.

### 3.4 Definition of the in-plane and out-of-plane models

The thesis will focus on the study of the seismic behaviour of masonry walls in the unreinforced and reinforced conditions. More specifically, the in-plane and out-of-plane failure mechanisms will be investigated adopting two different models of the wall. The topic of interest is to understand how these mechanisms change when one or more strengthening layers are implemented on the wall and how the layout of the reinforcement may influence them.

#### 3.4.1 In-plane model

In order to investigate the in-plane behaviour of a masonry wall a first model has been created. It consists of a wall with an opening and a lintel above it. The behaviour of the lintel is assumed to be elastic, thus the mechanical properties reported in Table 4 are assigned.

<table>
<thead>
<tr>
<th>Masonry Lintel Mechanical Parameters</th>
<th>LINEAR ELASTIC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\nu$ Poisson’s ratio</td>
<td>0.10</td>
</tr>
<tr>
<td>$E$ Young’s Modulus</td>
<td>3960 MPa</td>
</tr>
<tr>
<td>$\rho$ Density</td>
<td>1.90E-09 T/mm$^3$</td>
</tr>
</tbody>
</table>
The geometrical features of the in-plane model and its dimensions are reported in Figure 18. It has been considered a wall with a 0.2 m thickness and loaded apart from the self-weight with a distributed load deriving from a possible slab.

Figure 18 – In-plane model: Geometry and dimensions.

Different configurations are investigated, as represented in Figure 19: (a) the unreinforced wall and (b) the strengthened wall with TRM applied on the whole façade.

Figure 19 – In-plane models: unreinforced model and layout of the reinforced model.

The next step in the modelling procedure is the choice of the mesh size. Three different models have been created, respectively with a minimum mesh size of 25 mm, 50 mm, and 100 mm (Figure 20).
Modelling of the behaviour of TRM-strengthened masonry walls

Figure 20 – In-plane model mesh sizes: a) 25 mm; b) 50 mm; c) 100 mm.

Each model is characterized by the same load and boundary conditions. More specifically, the self-weight, applying the gravitational acceleration to the density of the material, and a distributed load are introduced. The value of the latter has been defined assuming a plan layout, as represented in Figure 21, and the presence of a slab oriented in the bigger dimension. Its weight has been supposed of about 1 kN/m. Consequently, the load has been divided in two equal areas, resulting in a distributed load of 2.4 N/mm on the 3.6 m walls. The load has been applied using 1-D elements to which a translational mass in the vertical direction is assigned.

Figure 21 – Plan layout.

Concerning the boundary conditions, the in-plane behaviour is simulated by fixing the translation in the horizontal (x-axis) and vertical (y-axis) direction on the base nodes. Moreover, all the nodes have been fixed in the out-of-plane (z-axis) direction as well as the rotations about the x and y directions, thus allowing only the rotation along the z-direction.

In order to select the best mesh size, an eigenvalue analysis and a push over analysis are performed on the models with different sizes.
Eigenvalue Analysis

Table 5 – Comparison of the principal frequencies/period and modes for different mesh size models.

<table>
<thead>
<tr>
<th># MODE</th>
<th>MODE SHAPE</th>
<th>MESH SIZE</th>
<th>FREQUENCY [Hz]</th>
<th>PERIOD [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1" alt="Mode 1 Shape" /></td>
<td>25 mm</td>
<td>45,447</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 mm</td>
<td>45,483</td>
<td>0.022</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100 mm</td>
<td>45,561</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td><img src="image2" alt="Mode 2 Shape" /></td>
<td>25 mm</td>
<td>88,595</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 mm</td>
<td>88,664</td>
<td>0.011</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100 mm</td>
<td>88,817</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td><img src="image3" alt="Mode 3 Shape" /></td>
<td>25 mm</td>
<td>117,376</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 mm</td>
<td>117,468</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100 mm</td>
<td>117,670</td>
<td></td>
</tr>
</tbody>
</table>

The results obtained running the Structural Eigenvalue Analysis in DIANA demonstrated that the mesh dimension has a null effect on both the principal frequency/period values and on the mode shapes. In fact, as it possible to observe in Table 5 the shapes obtained for the first three modes are exactly the same. Equally, the discrepancies between the values of the frequencies are extremely small. This difference is even smaller when converting the frequency to period.
Push-over Analysis

An ulterior demonstration of the fact that the mesh size is not influenced by the analysis results has been given by the Push-over analysis. With the purpose of estimating the sensitivity of the model when changing the length of the quadrilateral shells adopted, a simplified analysis has been done: the self-weight and the distributed loads have been disregarded while only a horizontal load has been applied. In Figure 22 the capacity curve obtained for each one of the models taken into account are presented.

![Capacity curve](image)

Figure 22 – Sensitivity analysis: Capacity curve.

As it possible to observe the curves corresponding to models characterized by different mesh sizes almost overlap each other. The maximum seismic coefficient for each one of them is almost the same and the difference between the obtained values (peak load, maximum displacement and load for the last converged step are reported for each one of the mesh sizes analysed in Table 6) is very small. More precisely, the curves slightly diverge from each other only in the post-peak branch.

Table 6 – Comparison of the Push over results for different mesh sizes (*Percentage error with respect to the results of the 100 mesh size).

<table>
<thead>
<tr>
<th>SENSITIVITY ANALYSIS</th>
<th>MESH SIZE [mm]</th>
<th>100</th>
<th>50</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>PEAK VALUE</td>
<td></td>
<td>1,366</td>
<td>1,352 (-1%)*</td>
<td>1,345 (-1,5%)*</td>
</tr>
<tr>
<td>LAST CONVERGED STEP</td>
<td></td>
<td>0,769</td>
<td>0,761 (-1%)*</td>
<td>0,757 (-1,5%)*</td>
</tr>
<tr>
<td>Maximum Displacement [mm]</td>
<td></td>
<td>0,9469</td>
<td>0,937 (-1%)*</td>
<td>0,932 (-1,5%)*</td>
</tr>
<tr>
<td>Seismic coefficient</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
As a consequence of what has been observed, the model with mesh size 100 mm is chosen for further analysis as it will be the most computationally cost effective.

### 3.4.2 Out-of-plane model

The out-of-plane behaviour of the wall object of this thesis has been studied elaborating a second model in which, apart from the façade, two transversal walls have been modelled. Its geometry and dimensions are represented in Figure 23.

![Figure 23 – Out-of-plane model: Geometry and dimensions.](image)

The characteristics of the out-of-plane model are exactly the same as in the previous in-plane case: thickness of the wall of about 0,2 m, loaded by the self-weight and the distributed load derived from the assumed roof structure (2,4 N/mm). Similarly, as in the in-plane study, apart from the unreinforced structure, two different strengthening configurations have been considered (Figure 24).

It is important to specify that for the finite element model only the 100 mm mesh size has been used.

![Figure 24 – Out-of-plane models: unreinforced, 1st layout (reinforcement applied on one side) and 2nd layout (reinforcement applied on both sides) of the wall (reinforcement is presented by grey colour).](image)
A specification for the out-of-plane reinforced model should be done, with regards to the modelling approach. In this case, instead of using the 8-nodes curved shells (CQ40S) for the masonry wall and the mortar matrix and the interface elements (CQ48I) in between them, 8-nodes curved layered shells are adopted (CQ40L, Figure 25). In these elements the thickness is subdivided into a number of layers. Each layer has its own material properties and is numerically integrated separately. This choice is due to the necessity to model the reinforcement layers applied to the outer and to the inner side of the wall.

![Figure 25 – Adopted elements in the Out-of-plane model: CQ40L.](image)

![Figure 26 – Layered elements: Definition of the layers and of the thickness.](image)

The definition of the elements thickness and layers requires the following input (Figure 26): the total thickness of the whole shell elements $t$ and the relative thickness of each layer which is defined by a factor $d_i$. Hence the thickness of each layer is defined as $t_i = d_i \times t$. The sum of the relative thickness $d_i$ should be equal to 1. Regarding the reinforcement, it is defined as a grid, like it was done in the in-plane models (Paragraph 3.3). The only difference is that in this case the grid will be assigned an eccentricity from the middle plane in which the layered shells are defined in order to locate it in the mortar layer. Following this approach, it is possible to define the reinforced models as follows.

<table>
<thead>
<tr>
<th>N° of layers</th>
<th>Thickness</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10 mm</td>
<td>MORTAR (Table 2)</td>
</tr>
<tr>
<td>2</td>
<td>200 mm</td>
<td>MASONRY (Table 1)</td>
</tr>
</tbody>
</table>

The reinforcement grid, which properties are defined in Table 3, is given an eccentricity of 105 mm (Figure 27-a).
Table 8-2\textsuperscript{nd} LAYOUT model - Reinforcement on both sides of the wall

<table>
<thead>
<tr>
<th>N° of layers</th>
<th>Thickness</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10 mm</td>
<td>MORTAR (Table 2)</td>
</tr>
<tr>
<td>2</td>
<td>200 mm</td>
<td>MASONRY (Table 1)</td>
</tr>
<tr>
<td>3</td>
<td>10 mm</td>
<td>MORTAR (Table 2)</td>
</tr>
</tbody>
</table>

Two reinforcement grid sets are created (properties in Table 3). To model the outer reinforcement an eccentricity of 105 mm in the positive direction is given and, similarly, for the inner reinforcement an eccentricity of 105 mm in the negative direction is assigned (Figure 27-b).

Figure 27 –Layered elements: Definition of the layers geometry: (a) 1\textsuperscript{st} layout and (b) 2\textsuperscript{nd} layout of the wall (reinforcement is presented by grey colour).
Modelling of the behaviour of TRM-strengthened masonry walls
4 IN-PLANE BEHAVIOUR OF THE MASONRY WALL

4.1 Introduction

In this chapter the in-plane behaviour of the unreinforced and the TRM-strengthened panels are compared by means of numerical analysis. The effectiveness of the Fibre reinforced concrete mortar is investigated in terms of load-displacement curves, compressive stresses, tensile strains and failure modes. Moreover, the model is studied when subjected to a dynamic load in terms of time history analysis. The strategies adopted for modelling the masonry wall in the unstrengthened and strengthened conditions are described in Chapter 3.

4.2 Unstrengthened masonry wall

The masonry wall is modelled, as described in the previous chapter, adopting a macro-modelling approach. The elements used for the panel are quadrilateral 8-nodes curved shells (CQ40S in DIANA) with a length size of 100 mm. The orthotropic behaviour of the material is simulated implementing a Rankine-Hill model, with different yielding function in compression (Hill-type) and in tension (Rankine-type). The parameters applied to this specific case of study are reported in Table 1 (Chapter 3). The lintel above the opening is assumed to have an elastic behaviour with the same elastic parameters of the masonry (Table 4 in Chapter 3). The adopted model is showed in Figure 28.

![Figure 28 – Unreinforced in-plane masonry model: Adopted FE mesh, loading and global coordinate system (the supports and the self-weight loading condition are not showed).](image)

The wall constrains simulate the in-plane behaviour, consequently all the nodes are fixed for the out-of-plane translation (z-axis) and rotations (around the x-axis and y-axis). Moreover, the two remaining translations are fixed in the nodes at the based. The structure is subjected to its own self-weight and a distributed load (2.4 N/mm) at the top deriving from a hypothetical slab. The response of the wall to a horizontal loading is first investigated in different compression levels.
4.2.1 Push-over analysis

The performance of the in-plane wall to a horizontal loading is investigated in two different vertical pre-compression conditions, namely when it is subjected only to its own weight and when it has distributed load in addition to its self-weight. A comparison is made between the performance of the wall in these two conditions by means of a push over analysis. More specifically, an incremental horizontal load proportional to the mass of the system is applied to the wall in the horizontal direction. As a point of control, where to read the results in terms of displacement in the horizontal direction, the node in the upper part of the wall (Node 2101) has been chosen (Figure 30). The reason behind this choice is that this is the position where, according to the mode shape corresponding to the principal frequency in the eigenvalue analysis, the maximum displacements are expected to take place.

![Figure 29 – Control point (node 2101) in the push over analyses.](image)

In this specific case of study, the explicit step size has a value of 0.02, while the convergence criterion applied is in terms of relative energy variation and has a value of 1.0E-03.

In Figure 30 the capacity curve of the in-plane model subjected to the incremental horizontal load are showed. In these curves it is possible to visualize the evolution of the load as a function of the displacement. It is important to specify that the distributed load on top of the wall is modelled adopting 1-D elements to which a translational mass in the vertical direction is assigned. This is crucial in order to take into account also the weight of the slab structure in the horizontal load, which is proportional to the mass. It can be observed that a higher level of pre-compression has an influence on the maximum load that the wall can sustain, as expected. In fact, while in the model in which only the self-weight (SW) is considered the peak load is about 37 kN, in the model in which also the distributed load is implemented (SW+DL) a peak load of about 55 kN is reached. Consequently, the presence of a higher value of vertical compression on the wall has led to a 49% higher horizontal capacity.

Analysing more in depth the results of the incremental analysis performed on the model under the effect of the self-weight and the distributed load, further observations can be done. As expected the deformed shape of the structure has the same features of the first mode of vibration (Figure 31) and the maximum displacement is reached in node 2101 (at the top of the wall).
Figure 30 – Capacity curve of the masonry wall subjected to a horizontal loading.

Specifically, in the step corresponding to the peak seismic coefficient of $\alpha=1.69$ (to which a base shear of 55kN corresponds) the maximum displacement reaches a value of about 0.342 mm in the horizontal direction.

Figure 31 – Contour map of the displacement in the horizontal direction at the peak seismic coefficient ($\alpha=1.69$) for the URM: Maximum displacement $d_x=+0.342$ mm.

In order to fully assess the behaviour of the structure, other two quantities should be evaluated, namely the principal compressive stresses and the principal tensile strains. The stresses allow to understand how the forces spread out into the wall and which is the load path they follow inside the
structure. On the other hand, mapping the strains is a way to highlight the areas where the cracks will appear.

In Figure 32 it is visible the diagonal compressed strut, which makes clear the path followed by the horizontal loading toward the constrains. The maximum value of the compression is understandably at the toe of the piers, and even if the range of compressive stresses we are dealing with is far from the values of the strength of the masonry (7.96% of the maximum compressive strength along the x-axis and 9.95% of the maximum compressive strength along the y-axis) it suggests which is the mode of failure of the wall under the effect of a horizontal load, namely a rocking motion. In fact, the plastic hinges are likely to develop where the maximum compression is reached.

![Contour map of the principal compressive stresses at the peak seismic coefficient (α=1.69) for the URM: Maximum compressive stress σ₃=0.796 MPa.](image)

It is possible to get to the same conclusion by observing the contour of the principle tensile strains. In Figure 33 it is displayed how the largest deformation occur at the base of the piers, exactly on the opposite side where the plastic hinges are supposed to be. This means that this is the location where the cracks will appear.

The strains results can be analysed more in depth taking into account the values range characterizing the structure. Knowing the Young’s modulus and the tensile strength of the masonry it is easy to derive the maximum tensile strain that the material can sustain. More specifically, in order to consider the parts of the structure in which the cracks are fully open, one may consider as a limit the tensile strain value corresponding to peak stress multiplied by a factor of 2 (Table 9). Therefore, in this specific case, as showed in Figure 33, the maximum principal tensile strain at the peak load is about 1.45e-03, meaning that the fully open crack limit has been extensively exceeded in both the x and the y directions.
4. IN-PLANE BEHAVIOUR OF THE MASONRY WALL

Figure 33 – Contour map of the principal tensile strains at the peak seismic coefficient (α=1.69) for the URM: Maximum tensile strain $\varepsilon_1=1.45\times10^{-3}$.

Table 9 – Masonry mechanical parameters: Maximum tensile strain.

<table>
<thead>
<tr>
<th>Masonry Wall Mechanical Parameters</th>
<th>MAXIMUM TENSILE STRAINS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ Young’s Modulus</td>
<td>3960 MPa</td>
</tr>
<tr>
<td>$\sigma_{tx}$ Tensile strength along x-axis</td>
<td>0.35 MPa</td>
</tr>
<tr>
<td>$\varepsilon_{tx}$ Maximum Tensile strain along x-axis</td>
<td>1.77e-04 MPa</td>
</tr>
<tr>
<td>Fully open crack along the x-axis</td>
<td>3.54e-04 MPa</td>
</tr>
<tr>
<td>$\sigma_{ty}$ Tensile strength along y-axis</td>
<td>0.25 MPa</td>
</tr>
<tr>
<td>$\varepsilon_{ty}$ Maximum Tensile strain along y-axis</td>
<td>1.26e-04 MPa</td>
</tr>
<tr>
<td>Fully open crack along the y-axis</td>
<td>2.52e-04 MPa</td>
</tr>
</tbody>
</table>

Figure 34 – Possible in-plane mechanism: localization of the plastic hinges.

Summing up, the results concerning the in-plane behaviour of the unreinforced masonry wall, object of this study, lead to the conclusion that it is characterized by a rocking motion mechanism, similar to the one represented in Figure 34.
4.2.2 Time History Analysis

In order to assess the seismic behaviour of the wall in dynamic conditions, a time history analysis has been performed adopting the DIANA solver Non Linear Transient Analysis. With this purpose a real accelerogram is chosen, namely the one measured during the L’Aquila (Italy) earthquake the 6th of April 2009. The data about the time history of the acceleration describing the earthquake have been taken from the ITACA database and then post-processed applying a filtering procedure in the software Seismosignal. Moreover, the displacement and velocity time history together with the Fourier and response spectra have been extracted.

*Loading*

The loading adopted for the transient non-linear analysis is derived from a 100s acceleration time history registered during the main-shock of L’Aquila earthquake in the station of L’AQUILA - VALLE ATERNO - CENTRO VALLE (AQV), (ITACA). As the highest values of acceleration were in the first 10s of the entire record, the duration of the base load implemented in the analyses has been reduced to 18s for computational cost efficiency. In Figure 35 the acceleration time history post-filtered in SEISMOSIGNAL 2016 (SEISMOSOFT) is presented.

![Figure 35 – L’Aquila main-shock - Amplitude Parameter, from top to bottom: Time Histories of Acceleration, Velocity and Displacement.](image-url)
To describe a ground motion in quantitative terms some parameters may be calculated (Oliveira, 2016). Usually, these quantities are defined in terms of amplitude, frequency content and duration.

The amplitude characteristics are defined in terms of time histories of acceleration, velocity and displacement. The standard procedure consists of recording one of them, usually the acceleration, and in obtaining the other two by integration. As it is possible to observe looking at the graph in Figure 35, while the acceleration is highly proportional to the high frequencies, the velocity and the displacement are dominated by lower frequencies as a consequence of the integration procedure. Moreover, other two measure useful to characterize a ground motion are the peak ground acceleration (PGA) and the peak ground velocity (PGV) which are correlated to the earthquake Intensity. The aforementioned quantities for the L’Aquila earthquake are summed up in Table 10.

![Graphs a, b, c, d](image-url)

**Figure 36 – L’Aquila main-shock - Frequency Parameter:** a) Fourier Amplitude spectrum, b) Elastic Response spectrum in displacement, b) Elastic Response spectrum in velocity, b) Elastic Response spectrum in acceleration.
Important information about an earthquake are also given by the frequency parameters. Among them, one of the most used is the Fourier Amplitude Spectrum, which describes how the amplitude of the ground motion is distributed with respect to the frequency. Also the Elastic Response spectra (in acceleration, displacement and velocity) are very often adopted for engineering applications. These spectra don’t describe the earthquake itself, but rather the maximum response of a SDOF system to a given seismic motion as a function of the frequency (or the period) and of the damping ratio of the system. The diagrams of the parameters above described for the L’Aquila earthquake are reported in Figure 36.

Also the duration plays a fundamental role in the description of an earthquake, mainly because the damages provoked on a structure, such as the strength degradation, are related to the number of load cycles to which the structure are subjected. Furthermore, the duration is a measure of the intensity of the seismic event, because it corresponds to the time needed to release the stored energy. As a consequence, earthquakes with a larger magnitude last longer.

Additionally, some other parameters, which can describe two or three characteristics (amplitude, frequency, duration) of the ground motion at the same time, may be also adopted (Table 10). Namely:

1) The Arias Intensity ($I_a$), which measures the potential destructivity of the earthquake. It is related to the duration and the acceleration of the ground motion;

2) The Cumulative Absolute Velocity (CAV), with which it is possible to describe the possible damages that the earthquake can produce;

3) The Spectral Intensity, defined as the integral of the pseudo-velocity over a period ranging from 0.1s to 2.5s. With this parameter it is possible to describe at the same time the amplitude and frequency content of the ground motion.

Table 10 – Ground Motion Parameters (Seismosignal, SEISTRUCT 2016)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_a$</td>
<td>2,80 m/s</td>
</tr>
<tr>
<td>SED</td>
<td>0,09 m$^2$/s</td>
</tr>
<tr>
<td>CAV</td>
<td>11,07 m/s</td>
</tr>
<tr>
<td>VSI</td>
<td>1,72 m</td>
</tr>
<tr>
<td>PGA</td>
<td>Max. Acceleration 0,62 g</td>
</tr>
<tr>
<td></td>
<td>Time of Max. Acceleration 2,14 s</td>
</tr>
<tr>
<td>PGV</td>
<td>Max. Velocity 0,36 m/s</td>
</tr>
<tr>
<td></td>
<td>Time of Max. Velocity 1,41 s</td>
</tr>
<tr>
<td>PGD</td>
<td>Max. Displacement 85,78 mm</td>
</tr>
<tr>
<td></td>
<td>Time of Max. Displacement 1,62 s</td>
</tr>
</tbody>
</table>
**Damping Factors**

In the transient non-linear analysis in order to consider the non-linearity of the material a damping model has to be defined. In this specific case, the Rayleigh Viscous Damping Model is adopted. It supposes that the viscous damping is distributed throughout the structure (Chopra, 1995).

This model assumes that the classical damping matrix $C$ is obtained as linear combination of the mass $M$ and the stiffness $K$ matrices:

$$C = \alpha M + \beta K$$  \hspace{1cm} (1)

Where, $\alpha$ and $\beta$ are two coefficients, which weight the contribution of the mass and the stiffness matrices of the structure in the definition of the damping. The values of $\alpha$ and $\beta$ are related to the damping ratios associated with the natural frequencies of the principal modes of vibrations of the structure. In general, they are calculated choosing two principal modes of vibration for which the damping ratios are known. However, when no information is available, the lower and the highest mode with a significant participation mass are adopted and values of the plausible damping coefficients are assumed. Then the (2), (3) and (4) are used.

$$\xi_n = \frac{1}{2} \left( \beta \omega_n + \frac{\alpha}{\omega_n} \right)$$  \hspace{1cm} (2)

$$\alpha = \xi \left( \frac{2 \omega_i \omega_j}{\omega_i + \omega_j} \right)$$  \hspace{1cm} (3)

$$\beta = \xi \left( \frac{2}{\omega_i + \omega_j} \right)$$  \hspace{1cm} (4)

The matter of the choice of the $i$-th and $j$-th natural frequencies to be used in the equations showed above is crucial in order to characterize correctly the dissipative behaviour of the structure also for the modes lying in between them. The frequencies of the selected modes should be in the linear range in order to consider the structure in an undamaged state (Chopra, 1995). For the model of the in-plane wall, the 1$^{st}$ and the 6$^{th}$ modes are selected (Table 11).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Cumulative Mass Participation [%] in the x-direction</th>
<th>$\omega$ [rad/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45,56</td>
<td>50,6%</td>
<td>286,3</td>
</tr>
<tr>
<td>6</td>
<td>162,06</td>
<td>61,6%</td>
<td>1018,3</td>
</tr>
</tbody>
</table>

Table 11 – Modes used for the calculation of the Rayleigh Damping.
Usually the adopted modes should activate a cumulative participating mass of about 85%. However, in this case, as the highest modes contribute with an extremely small participating mass, the 6th mode is used and a mass participation higher than 60% is considered acceptable. A damping ratio of 2% has been assumed for the selected modes. The values of the Rayleigh coefficients are then derived as explained above, obtaining $\alpha = 8.9379$ and $\beta = 0.00003$. Figure 37 shows a graphical representation of the distribution of the damping ratios in between the 1st and the 6th modes. As it possible to note, the values of $\xi$ range between 1.5% and 2%, which is a very narrow interval. Usually the structural behaviour is not very sensitive to these small variations.

![Figure 37](image_url) - Variation of the damping ratios along the modes.

**Time Integration Method**

When performing a non-linear analysis where a dynamic load is applied, the selection of an appropriate time integration method is of extreme importance. The determination of an analytical solution for the response of a structure subjected to an earthquake (acceleration time history) is not possible because of the high variability of the excitation during the time and because of the non-linear behaviour of the structure. Such a problem can be overcome numerically, performing a time integration of the differential equations. In general, a numerical method should be able to fulfil three criteria:

1) **Convergence**: decreasing the size of the time step $\Delta t$ the numerical solution should approach the exact solution;

2) **Stability**: the numerical solution should remain stable in the presence of numerical round-off errors.

3) **Accuracy**: the results of the numerical solution should be close enough to the real solution.
The numerical methods which could be adopted in the time-stepping procedures belong to two main categories: *Explicit and Implicit methods*.

In the Explicit method, the unknown parameter \( u_{i+1} \), the response of the structure at the instant \( t_{i+1} \), depends on \( t_i \). Thus all the unknown parameters can be explicitly computed without solving the equation of motion at the instant \( t_{i+1} \). The drawback of this method is that it becomes unstable for large time steps \( \Delta t = t_{i+1} - t_i \).

On the other hand, the Implicit method evaluates the response of the structure \( u_{i+1} \), solving the equation of motion at the instant \( t_{i+1} \). Choosing properly the parameters \( \alpha \) and \( \gamma \), which characterizes the integration method associated with this method, it is possible to obtain an unconditionally stable solution of the system. As in the explicit method, it is recommended, in order to obtain accurate results, to choose small time steps. This last method is adopted in the non-linear dynamic analyses carried out in this thesis.

Among the available implicit methods, the Hilber-Hughes-Taylor (HHT) method is used because it allows to solve one problem that can occur while studying masonry structures under the effect of a dynamic load. These type of structures are, in fact, characterized by a quasi-brittle behaviour, hence the sudden change of the properties from linear to fully cracked state could be the cause of a numerical noise (Cervera et al., 1995). The HHT model implements a damping for this kind of noise without affecting the accuracy of the solution (Faria, 1994).

The finite difference equations for the HHT are the following:

\[
\dot{u}^{t+\Delta t} = \dot{u}^t + [(1 - \gamma)\ddot{u}^t + \gamma \dot{u}^{t+\Delta t}]
\]

\[
u^{t+\Delta t} = u^t + \dot{u}^t \Delta t \left[ \left( \frac{1}{2} - \beta \right) \ddot{u}^t + \beta \dot{u}^{t+\Delta t} \right] \Delta t^2
\]

where

\[
\gamma = \frac{1}{2} (1 - 2\alpha)
\]

\[
\beta = \frac{1}{4} (1 - \alpha)^2
\]

Here \( u, \dot{u}, \ddot{u} \) are respectively the displacement, the velocity and the acceleration, while \( \gamma \) and \( \beta \) are parameters associated with the Newmark’s method and \( \alpha \) is a parameter specifically related to the HHT method (if \( \alpha = 0 \) the method reduce to the Newmark’s method). \( \Delta t \), as mentioned before, is the
time step. Consequently, using the (5), (6) the equations of motion of a multi-degrees-of-freedom (MDOF) system (9) becomes the expression (10):

$$
\mathbf{M}\ddot{\mathbf{u}}^{t+\Delta t} + \mathbf{C}\dot{\mathbf{u}}^{t+\Delta t} + \mathbf{K}\mathbf{u}^{t+\Delta t} = f_{ext}^{t+\Delta t}
$$

(9)

$$
\mathbf{M}\ddot{\mathbf{u}}^{t+\Delta t} + (1 + \alpha)\mathbf{C}\dot{\mathbf{u}}^{t+\Delta t} - \alpha\mathbf{C}\dot{\mathbf{u}}^{t} + (1 + \alpha)f_{int}^{t+\Delta t} - \alpha f_{int}^{t} = (1 + \alpha)f_{ext}^{t+\Delta t} + \alpha f_{ext}^{t}
$$

(10)

where

$$
f_{int} = K\mathbf{u}
$$

(11)

We define $f_{int}$ and $f_{ext}$ as the vectors for the restoring and the external loads respectively. The HHT is defined as a second order accurate and unconditionally stable method. When adopting this approach, it is fundamental to choose accurately the values for the parameters $\alpha$, because it is indirectly proportional to the damping of the numerical noise. Acceptable values for this coefficient range between $-1/3$ and $1/2$. Usually a higher damping is adopted for higher frequencies and lower damping for low frequency modes. In the solution of the non-linear dynamic analysis the default values of the software DIANA (TNO) are adopted.

Another important variable to define in order to obtain accurate results is the time step size $\Delta t$. Its definition derives from two considerations:

1) The dimension should be much smaller than the duration of the entire dynamic load $t_d$;

2) The dimension of the step size can be obtained from the mode with the lowest frequency and the highest period as follow:

$$
\Delta t = \frac{1}{10}T_i
$$

(12)

In this specific case of study the time step is calculated as reported in Table 12. The value obtained from the calculation is rounded to $\Delta t=0,0025$ s, which also satisfy the first condition. In fact, the total duration of the dynamic load is $t_d=18$ s $>> \Delta t=0,0025$ s.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Period [s]</th>
<th>Time step [s]</th>
<th>Time step adopted [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45,56</td>
<td>0,0219</td>
<td>0,00219</td>
<td>0,0025</td>
</tr>
</tbody>
</table>
Furthermore, in each step of the non-linear dynamic analysis an iterative method is used to reach the equilibrium condition. More specifically, the Secant method with a convergence criterion based on the energy and a tolerance of 0.001 has been implemented.

Results

The unreinforced in-plane model has been studied under the effect of the accelerogram showed in Figure 35 applied in the horizontal direction (x-axis). In order to have a more complete study of the dynamic response of the structure several dynamic non linear analysis have been performed, changing the seismic coefficient. First the accelerogram is applied as it is (L'Aquila Earthquake 100%) and then amplified with a factor of 2 (L'Aquila Earthquake 200%) and with a factor of 3 (L'Aquila Earthquake 300%).

The response of the structure in all the cases is presented in terms of principal tensile strains, displacement and base shear time histories, relative displacements, storeys drifts and hysteretic behaviour of the wall.

Cracking pattern (principal tensile strains)

In Figure 38 the damage of the structure produced by the dynamic load is presented in terms of principal tensile strains. The contouring map presented is a graphical representation of the maximum tensile strains occurred during the whole earthquake. This plotting has been obtained by searching in iDiana (DIANA TNO) the most severe conditions of damage among all the time steps of the dynamic analysis.

The maps show that under the effect of the three load cases considered (100%, 200% and 300% of the L'Aquila earthquake) the highest tensile strains are concentrated at the base of the piers, on the lintel and at the top of the wall.

In the first case (Figure 38-a) the maximum tensile strain is $\varepsilon_1=4.12e-05$, which is lower than the limit of cracking $\varepsilon_c=8.8e-05$ (Table 9). Hence it is possible to conclude that the structure is not in a cracked condition. On the contrary, when an earthquake with a doubled amplitude is applied the damage grows. In fact, although the strain concentration is similar to the previous case, the values are increased and the highest value of the tensile strain is $\varepsilon_1=2.38e-04$. Thus, not only the cracking limit but also the fully cracked limit ($\varepsilon_{f_{cr}}=1.26e-04$) have been overcome. In the third load condition (300%), even more so, the fully cracked limit value of the tensile strains is overcome (the extreme value of the tension in this case is $\varepsilon_1=1.23e-03$). The more the earthquake intensity increases the more the strains are concentrated at the base of the piers (Figure 38-c).

These results are coherent with the conclusion given by the push over analysis, where a peak seismic coefficient of 1,689 was obtained. Hence, it was expected that a dynamic load with a PGA of 0,6231g (load case 100% L'Aquila Earthquake) wouldn't cause an extreme damage. As for the crack pattern, the maps obtained are very similar (Figure 39), but some considerations should be made. While in the push over the load is monotonic in the dynamic analysis the load is cyclic, thus the cracking occurs on
both sides of the piers. Considering that the structure is symmetric it is plausible that the crack pattern would be the same but mirrored when the monotonic load would be applied in the opposite direction (Figure 39 -b).

Figure 38 – Cracking of the in-plane unreinforced wall in terms of principal tensile strains: a) under the effect of L’Aquila earthquake (100%); b) under the effect of L’Aquila earthquake (200%); c) under the effect of L’Aquila earthquake (300%).
Figure 39 – Comparison of the cracking pattern of the in-plane unreinforced wall obtained from different analysis: a) results of the dynamic analysis under the effect of L’Aquila earthquake (300%), b) results of the push over analysis (peak seismic coefficient α=1,689).

Base Shear Time Histories and Hysteretic behaviour

Another meaningful output of the dynamic analysis is the base shear, particularly its evolution in the period of time during which the dynamic base load is applied. The estimation of this quantity is a measure of the total load to which the structure is subjected. As it is represented in Figure 40 the values of the sum of the horizontal reactions in the case of the 300% earthquake are higher, as it was expected. More specifically, in this case the maximum value of the base shear registered is about 44,8 kN, which is comparable to the weight of the structure (32,9 kN).

Figure 40 – Time Histories of the base shear: (in blue) under effect of L’Aquila earthquake (100%), (in orange) under the effect of L’Aquila earthquake (200%), (in red) under effect of L’Aquila earthquake (300%).

With the purpose of understanding the behaviour of the structure when subjected to a dynamic load, thus to cycles of loading and unloading, it is of more interest the description of the evolution of the base shear with respect to the displacement in the same direction. To this extent the hysteretic response of the structure, compared with the pushover results is depicted in Figure 41 and Figure 42.
Figure 41 – Comparison of the Base shear-displacement results in the x-direction with the push over results: (a) under the effect of L’Aquila earthquake (100%), (b) under the effect of L’Aquila earthquake (200%), (c) under the effect of L’Aquila earthquake (300%).
Figure 42 – Push-over curve and points of maximum displacement, and in this case also of maximum shear, in the dynamic analyses (in blue- L’Aquila earthquake 100%, in orange- L’Aquila earthquake 200%, in red- L’Aquila earthquake 300%).

The curves plot the base shear in terms of seismic coefficient against the relative displacement of the same point control (node 2101) at the top of the wall adopted in the push-over analysis (Figure 43). It can be observed that, as expected, the hysteretic curve when the dynamic load is higher shows a non-linear behaviour. On the contrary in the first two cases (Figure 41 a- b) the structure is still behaving in the linear field or with very little nonlinearity. Furthermore, in Figure 42 a comparison of the output of the push-over and of the dynamic analysis is represented: the points corresponding to the maximum base shear coefficient and to the maximum displacement are plotted together with the capacity curve. It results that the push over analysis overestimates the values both in terms of force and of displacements.

*Displacement Time Histories and Relative displacements in elevation*

The values of the relative (with respect to the base of the wall) displacements for some points of the wall along all the time duration of the loading are extracted from the results of the non-linear dynamic analyses.

Figure 43 – Nodes selected for the output of the non-linear dynamic analyses.
As the total height of the structure is only 2.4 m and the effect of an earthquake are amplified when the height increases, the relative displacements between the selected nodes and the base are limited. The maximum relative displacement however is reached in the nodes at the top (node 937 and node 2101). As an example, the time histories of the displacements of the node 2101 in the different load cases are represented in Figure 44.

Apart from the evident difference in value of the relative displacement in the more severe load case (maximum displacement of about 0.36 mm), another aspect arises from the comparison of the three time-histories in Figure 44, namely the phenomena occurring in the 200% and 300% load cases, where the equilibrium position is lost after the first seconds. This fact can be explained as a consequence of the development of irreversible deformations. Hence the structure is not able to return to the undeformed condition. This observation confirms what already deduced from the study of the hysteretic behaviour, where only in the 300% accelerogram the structure behaved non-linearly (Figure 41-c).

![Figure 44](image)

**Figure 44 – Relative-displacement of node 2101 in the x-direction: (in blue) under the effect of L’Aquila earthquake (100%), (in orange) under the effect of L’Aquila earthquake (200%),(in red) under the effect of L’Aquila earthquake (300%)**

From the entire displacement time-histories of the other selected nodes, more specifically the nodes that lie along the same vertical alignment (node 889, node 925 and node 937 - Figure 43), the maximum relative displacements are extracted. Then, these maximum values are plotted against their height from the base level. The observation that arise from the comparison of the results in terms of relative displacements, obtained from different types of analyses (dynamic and push-over), is that the values given by the non linear static analysis are similar to the results given by the dynamic analysis in which the accelerogram with the highest seismic coefficient (300%) is assigned. More specifically in Figure 45 the profiles of the maximum relative displacements and maximum relative displacements divided by the node height (drift) from the ground floor are plotted.
4. IN-PLANE BEHAVIOUR OF THE MASONRY WALL

4.3 TRM-strengthened masonry wall

The model of the wall is now investigated with the addition of two TRM composite strengthening layers on both its sides. With this purpose, the analyses are performed considering half of the whole model: only with one reinforcement layer and a wall with half of the thickness (100 mm) of the previous case are modelled. In Figure 46 the finite element model is showed.

![Finite element model](image)

Figure 46 – TRM-reinforced masonry model: Adopted FE mesh and loading (the supports and the self-weight are not showed).

Figure 45 – Relative-displacements in elevation: (a) Profile-relative displacement against the height of the structures; (b) Drift against the height of the node.
The characteristics of the masonry material panel are exactly the same as in the unreinforced case previously presented. On the other hand, the mortar matrix of the reinforcement is modelled, as described in Chapter 3, with 8 nodes curved shell (CQ40S in DIANA) to which a Total strain crack based model has been applied. The specific properties are reported in Table 2 (Chapter 3). The reinforcement, instead, is simulated by introducing a grid reinforcement (Table 3) embedded in the mortar with a perfect bond. Furthermore, between the masonry wall and the strengthening layer a stiff interface is created in order to prevent any relative displacements.

As in the unreinforced model an in-plane behaviour is simulated introducing specific boundary conditions on the masonry wall: at the base nodes the translations in the $x$ and $y$ direction are fixed, while all the nodes are restrained against the out-of-plane translations ($z$-axis) and rotations (around the $x$ and $y$ axis). Moreover, for what it concerns the loading cases, the self-weight and a distributed load are computed. More specifically, as we are considering half a model, its value is 1.2 N/mm (half of the load computed in the unreinforced model) assigned to the masonry panel by means of 1-D elements to which a translational mass in the vertical direction is attributed.

### 4.3.1 Push over Analysis

As in the previous case the performance of the reinforced structure to a horizontal loading is studied by means of a Push-over analysis. The results then are presented in terms of seismic coefficient-displacements curves, principal tensile strains and principal compressive stresses. The push-over curve is drawn considering the horizontal displacement of the same control point (Figure 29) at the top of the wall (Node 2101) as in the previous case. The analysis has been performed for different types of reinforcement (PBO, Basalt and Glass). Their properties are defined in Table 3 in Chapter 3.

The analysis is performed adopting a Newton-Raphson Regular iteration method with an explicit step size of 0.02 and a convergence criteria applied in terms of relative energy variation of $1.0 \times 10^{-3}$.

In Figure 47 the response curves of the reinforced in-plane model for different types of reinforcement are compared with the behaviour of the unstrengthen structure. As it is possible to observe, the implementation of the TRM composites allow the structure to sustain higher horizontal seismic coefficients. The analysis concerning the reinforced models showed some convergence problems, and the analysis could be run until the steps displayed in the graph. The convergence problem could be related to the approaching to the peak load, thus in this cases the results reported are in the last converged step. Nevertheless, from the results obtained it is already clear that the behaviour of the structure, in terms of capacity seismic coefficient and displacements (the reinforced structure is more ductile than the unreinforced), improves largely.

As the results in Table 13 shows, among the different typologies of fibres adopted, the maximum increase is obtained with the PBO material ($\alpha=5.03$ with an increase of about 174.86% from the Unreinforced wall being $\alpha=1.83$).
4. IN-PLANE BEHAVIOUR OF THE MASONRY WALL

Figure 47 – Capacity curve of the TRM-strengthened masonry wall (in-plane) subjected to a horizontal loading proportional to the mass.

Table 13 – Push over results: Seismic coefficient for unreinforced and reinforced models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Unreinforced wall</th>
<th>Reinforced wall</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Seismic coefficient (URM)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic coefficient at the last converged step (RM)</td>
<td>[g] 1,69</td>
<td>6,11</td>
<td>5,19</td>
</tr>
<tr>
<td>Increase of the Seismic coefficient</td>
<td>-</td>
<td>261,6%</td>
<td>207,3%</td>
</tr>
</tbody>
</table>

For what it concerns the displacements, the map contouring plot of the PBO-reinforced wall are reported in Figure 48. For the models reinforced with basalt and glass the output are qualitatively the same. As it possible to observe the deformed shape has not changed from the unreinforced case, but the value of the maximum displacement is higher, meaning that the behaviour is more ductile.

Figure 48 – Contour map of the displacement in the horizontal direction at the peak seismic coefficient (α=6,11) for the PBO-RM: Maximum displacement $d_x=+3,247$ mm.
Furthermore, the results in terms of principal compressive stresses (in the masonry) and principal tensile strains (in the mortar) are presented in Figure 49 and Figure 50. The extreme values of the stresses and strains are concentrated at the base of the piers and at the corner of the openings. The absolute values of the principal compressive stresses and tensile strains are higher in the reinforced model than in the unreinforced. The first one is due to the higher load capacity of the reinforced façade, while the second one could be explained as a widespread distribution of the strains resulting from the possibility to dissipate more energy with less strains concentration.

The maximum value of the compression in the model reinforced with PBO is about $\sigma_3 = -3.361$ MPa, which is however the 33.6% of the compressive strength along the x-axis (10 MPa) and the 42% of the compressive strength along the y-axis (8 MPa).

The maximum value of the tensile strains in the mortar is about $\varepsilon_1 = 3.409\times10^{-3}$ which is higher than the cracking limit of the mortar (7.917\times10^{-4}). Hence, this is the component of the system where the failure is happening.

Furthermore, the stresses in the reinforcement are presented in Figure 51 and Figure 52. The concentration of the extreme stresses corresponds to the critical areas observed in the mortar matrix (corner of the opening, top of the wall and base of the piers). This is because the tensile stresses are being transferred from the mortar, which is failing, to the fibre reinforcement. The largest values in both directions ($\sigma_{xx} = 161.12$ MPa and $\sigma_{yy} = 639.44$ MPa) are very far from their tensile strength (3400 MPa).

![Contour map of the principal compressive stresses at the peak seismic coefficient (\(\alpha=6.11\)) for the PBO-RM: Maximum compressive stress \(\sigma_3 = -3.361\) MPa.](image)

Figure 49 – Contour map of the principal compressive stresses at the peak seismic coefficient (\(\alpha=6.11\)) for the PBO-RM: Maximum compressive stress \(\sigma_3 = -3.361\) MPa.
Figure 50 – Contour map of the principal tensile strains at the peak seismic coefficient (α=6.11) for the PBO-RM (in the mortar): Maximum tensile strain $\varepsilon_1 = 3.409e-03$.

Figure 51 – Contour map of the tensile stresses along the x-local axis at the peak seismic coefficient (α=6.11) for the PBO-RM (in the reinforcement): Maximum tensile stress $\sigma_{xx} = 161.12$ MPa.
Figure 52 – Contour map of the tensile stresses along the γ-local axis at the peak seismic coefficient ($\alpha=6.11$) for the PBO-RM (in the reinforcement): Maximum tensile stress $\sigma_{yy}=639.44$ MPa.

4.3.2 Time History Analysis

The seismic response of the TRM-strengthened wall is now studied, similarly to what have been done in the unreinforced case, in dynamic conditions. More specifically, only one typology of reinforcement has been chosen, namely the TRM composite with glass fibres. The time history analysis has been performed with DIANA software, running the solver Non Linear Transient Analysis. The loading is a real accelerogram obtained from the registration of the L’Aquila earthquake.

Table 14 – Comparison between the most participating modes in the unreinforced and reinforced models.

<table>
<thead>
<tr>
<th></th>
<th>UNREINFORCED</th>
<th>REINFORCED</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td># Mode</td>
<td>Frequency [Hz]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>45.56</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>162,06</td>
</tr>
</tbody>
</table>

The details about the time integration method implemented in the software and the information concerning the seismic event are the same as described in Paragraph 4.2.2. Moreover, regarding the damping factor for the non-linear materials, the same parameters as in the unreinforced model are assigned ($\alpha=8.9379$ and $\beta=0.00003$). In fact, as the additional layer of the reinforced mortar doesn’t change significantly the mass of the wall, the behaviour of the structure in terms of frequency is almost the same. More specifically, the first and the last most participating modes (Table 14) have similar frequencies, hence there is a negligible difference in the Rayleigh damping factors calculated with the equations (2), (3), (4) previously introduced.
Results

The seismic response of the reinforced structure is evaluated with an incremental dynamic analysis. In other words, the transient non-linear analysis has been repeated three times, increasing the seismic coefficient with which the L'Aquila accelerogram is assigned to the model (100%, 200% and 300%). This procedure allows to obtain a more thorough understanding of the behaviour of the structure. The response of the structure in both cases is presented in terms of principal tensile strains, displacement and base shear time histories, relative displacements, storeys drifts and hysteretic behaviour of the wall.

Cracking pattern (principal tensile strains)

In Figure 53 it is presented the contour plot of the maximum principal tensile strains occurred during the whole duration of the accelerograms.

Figure 53 – Cracking of the in-plane reinforced wall in terms of principal tensile strains: a) under the effect of L'Aquila earthquake (100%); b) under the effect of L'Aquila earthquake (200%); c) under the effect of L'Aquila earthquake (300%).
They are obtained in the software iDiana (DIANA TNO) scanning all the time steps and searching the worst condition among them. This procedure allows to highlight the areas of the wall where the cracking appears.

As it was already observed in the push over analyses, the behaviour of the structure, in terms of failure mechanism doesn’t change from the unreinforced case. Also in this case, the extreme values of the tensile strains are located in all the load cases (100%, 200% and 300% of L’Aquila earthquake), at the base of the piers, on the lintel and on top of the wall. However, the peaks are registered at the base of the wall, thus it is possible to conclude that a rocking motion is likely to occur.

Another important consideration is that in the reinforced structure, while the masonry wall works mainly in compression, the tensile stresses induced by the horizontal loading are transferred to the mortar, which in turn transfers them to reinforcement grid. As a consequence, the strengthened wall is able to resist to higher levels of loading.

In the first loading condition (100%), the maximum value of tensile strains is $\varepsilon_1=4.24\times10^{-5}$, which is comparable with the peak obtained in the same loading conditions for the unreinforced model ($\varepsilon_1=4.12\times10^{-5}$). When the earthquake with factor 2 is applied the tensile strains increase and the maximum value is $\varepsilon_1=1.93\times10^{-4}$, while in the the unreinforced model was higher $\varepsilon_1=2.38\times10^{-4}$. For the last case, seismic coefficient 3, the peak is $\varepsilon_1=4.13\times10^{-4}$, which is much lower than the corresponding value in the unreinforced case ($\varepsilon_1=1.23\times10^{-3}$). From the comparison of the values of the tensile strains in the in-plane model for the two conditions evaluated (unreinforced and reinforced) it is clear that the effectiveness of the TRM-strengthening technique becomes clear and clear while increasing the intensity of the earthquake. In the 100% load case the earthquake doesn’t cause damages to the masonry structure (the fully cracked limit $\varepsilon_0=8.8\times10^{-5}$, Table 9) is not exceeded and as a consequence the implementation of the reinforced layer is not needed. On the contrary, when the seismic coefficient grows the differences in terms of tensile strains increases too (Figure 54). In this case the tensile strains are being transferred to the reinforced mortar, and as a consequence the performance of the wall is enhanced.

![Figure 54 – Maximum tensile strains in the unreinforced and reinforced model for different earthquake seismic coefficient.](image)

In conclusion, the TRM strengthening technique is effective in enhancing the seismic resistance of masonry walls, especially when the intensity of the earthquake increases.
Base Shear Time Histories and Hysteretic behaviour

Figure 55 – Comparison of the Time Histories of the base shear in the unreinforced (grey) and reinforced models: (a) under effect of L’Aquila earthquake (100%), (b) under the effect of L’Aquila earthquake (200%), (c) under effect of L’Aquila earthquake (300%).
As it has been done for the unreinforced model, the evolution of the base shear in the period of time during which the dynamic base load is applied is presented. This kind of output is important because it allows to estimate the total load to which the structure is subjected. It is obtained as the sum of the horizontal reactions at the base of the structure.

In Figure 55 the different time histories for the unreinforced and reinforced models are presented for each one of the load cases studied. It is clear that the values of the base shear become higher and higher the more the seismic coefficient is increased. Moreover, in each case, a reduction of the seismic load is observed when the strengthening is introduced. In more details, the maximum value of the base shear is registered in the 300% loading case and it is 44,8 kN in the unreinforced model, which is higher than the weight of the structure (32,9 kN). However, when the TRM-strengthening layer is introduced the highest base shear is about 26,2 kN. An ulterior confirmation of the effectiveness of TRM-composite strengthening is obtained. In all the dynamic analysis performed a reduction higher then the 40% is obtained (Table 15).

Table 15 – Comparison between base shear values in the unreinforced and reinforced models for different seismic coefficients.

<table>
<thead>
<tr>
<th>MODEL</th>
<th>BASE SHEAR</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Seismic coefficient</td>
<td>Seismic coefficient 2</td>
<td>Seismic coefficient 3</td>
</tr>
<tr>
<td>UNREINFORCED</td>
<td>17,4 kN</td>
<td>34,2 kN</td>
<td>44,9 kN</td>
</tr>
<tr>
<td>REINFORCED</td>
<td>9,7 kN</td>
<td>19,1 kN</td>
<td>26,2 kN</td>
</tr>
</tbody>
</table>

As the dynamic load assigned is characterized by cycles of loading and unloading, it is interesting to describe the performance of the models in terms of hysteretic behaviour. Hence in Figure 55 the evolution of the shear coefficient in relation to the relative displacement of a node at the top of the wall (node 2101, Figure 43) is depicted for the different seismic coefficients. The results are compared with the equivalent load cases in the unreinforced model and with the push over curve for the glass fibres reinforced model. The comparison of the hysteretic behaviour shows that the reinforced model has a slight non-linear behaviour only in the case with seismic coefficient 3 (Figure 56). However, an enhancement of the response is visible. In fact, the value of the shear coefficient is much lower and the non-linearity is not so significant as it was in the unreinforced case. Furthermore, a comparison of the dynamic results with the push over analysis results are presented (Figure 57), and as it was observed in the unreinforced model the push over overestimates both the maximum displacement and the maximum shear force.
Figure 56 – Comparison of the Base shear-displacement results in the x-direction with the push over results: a) under the effect of L’Aquila earthquake (100%), b) under the effect of L’Aquila earthquake (200%), c) under the effect of L’Aquila earthquake (300%).
Modelling of the behaviour of TRM-strengthened masonry walls

Figure 57 — Push-over curve and points of maximum displacement, and in this case also of maximum shear, in the dynamic analyses (in blue - L'Aquila earthquake 100%, in orange - L'Aquila earthquake 200%, in red - L'Aquila earthquake 300%).

Another aspect that the graphs in Figure 57 highlight is that, even if the maximum values of the absolute displacement in the unreinforced and reinforced models are the same, the maximum shear coefficient corresponding to them is different. More specifically, it is lower when the TRM-strengthening layer is added.

Displacement Time Histories and Relative displacements in elevation

The values of relative (with respect to the base of the wall) displacements for some points of the wall along all the time duration of the loading are extracted from the results of the non linear dynamic analyses. The nodes selected for these output are in the same position as in the unreinforced model (Figure 43). In Figure 58 the time histories in different loading conditions of the relative displacement of the node 2101 is represented.
As it was observed in the output previously described, the differences in the response of the unreinforced model and the reinforced one increase as the intensity of the earthquake grows. The maximum relative displacements are, also in this case, obtained at the node in the upper part of the wall (node 937 and node 2101). The maximum relative displacement, which is clearly obtained when the accelerogram with seismic coefficient 3 is applied, is about 0.36 mm, the same as it was in the unreinforced model. However, even if it is not possible to observe a difference in terms of highest relative displacement, the behaviour of the wall in the unstrengthened and strengthened conditions differs, in terms of irreversible deformations, when the earthquake intensity increases.

In Figure 59 it is represented the relative displacement of the node 2101, for the three load cases, in the two conditions in which the wall is studied.

![Graph](image-url)

**Figure 59** – Comparison of the Relative-displacement of node 2101 in the x-direction in the unreinforced and reinforced models: (a) under effect of L’Aquila earthquake (100%), (b) under the effect of L’Aquila earthquake (200%), (c) under effect of L’Aquila earthquake (300%).
While in the case (Figure 59-a,b) with seismic coefficient 1 and 2 no difference can be observed, in the third case (Figure 59-c) it is evident as in the reinforced model the value of the irreversible deformations reduce from the unreinforced case.

Hence, it is demonstrated, once again, that the presence of the TRM-layer has more importance when the intensity of the earthquake is higher.

Lastly, in Figure 60, the profiles and the drifts for the reinforced wall are presented. The displacements are read in the nodes that lie along the same vertical alignment (node 889, node 925 and node 937 - Figure 43) as in the unreinforced model. Then, the maximum relative displacements, extracted from the whole time histories, are plotted against their height from the base. For the drift, these values are expressed as a percentage of the height of the node from the ground level.

![Graph showing relative displacements and drifts](image-url)

**Figure 60 – Relative-displacements in elevation:** (in blue) under effect of L’Aquila earthquake (100%), (in red) under the effect of L’Aquila earthquake (200%), (in green) under the effect of a monotonic load proportional to the mass of the wall-push over analysis.

The results obtained for the time histories (100%, 200% and 300%) are also compared to values of displacement resulting from the push over analysis in correspondence of a seismic coefficient ($\alpha = 1.77$) comparable to the maximum PGA acceleration considered (the one in the time history with a seismic coefficient of 3).
5 OUT-OF-PLANE BEHAVIOUR OF THE MASONRY WALL

5.1 Introduction

This chapter is about the comparison of the out-of-plane performances of unstrengthened and TRM-strengthened walls by means of numerical analysis. The results of the analysis carried out are presented in terms of load-displacement curves, compressive stresses, tensile strains and failure modes. The modelling strategies adopted for both models are as described in Chapter 3.

5.2 Unstrengthened masonry wall

The unreinforced configuration of the out-of-plane masonry model is set up adopting, as specified in Chapter 3, a macro-modelling approach. As in the previous in-plane case, the masonry structure is reproduced with quadrilateral 8-node curved shells (CQ40S element in DIANA) with a length size of 100mm. The behaviour of the material is simulated implementing a Rankine-Hill model. The specific parameters applied are reported in Table 1 (Chapter 3). For what it concerns the lintel above the opening, it is assumed to behave in an elastic field, and the elastic parameters (Elastic modulus, Poisson's ratio and density) assigned are the same as the masonry material (Table 4 in Chapter 3). The adopted model is showed in Figure 61.

Figure 61 – Unreinforced out of plane masonry model: Adopted FE mesh, loading and global coordinate system (the self weight loading condition is not showed).

The boundary conditions of the model have been defined as follow: the nodes at the base are fixed for all the translations (x, y, z-axes). Moreover, for the nodes of the transversal walls the translations in
the horizontal direction (x-axis) are not allowed. Two gravitational loading conditions are defined: the self-weight and a distributed load (2.4 N/mm) at the top deriving from a hypothetical slab. The distributed load is defined by applying translational masses in the vertical direction. In the continuation of the thesis, the seismic performance of the structure is investigated.

5.2.1 Push-over analysis

The seismic out-of-plane behaviour of the model described in the previous paragraph is assessed by means of a non-linear static analysis (push over analysis). The load, proportional to the mass of the structure, is applied in the z-direction and it is monotonically increased. The control point chosen to read the results in terms of out-of-plane displacements, and consequently to obtain the load-displacement curves, is the node 1057 located at the middle of the upper part of the façade (as displayed in Figure 62). In fact, this is the node where the maximum displacements occur.

![Figure 62 – Point of control for the Push over Analysis in the out of plane model.](image)

The pushover analysis is performed by application of the load proportional to the structure’s mass obtained by assigning an acceleration in z direction. As in the in-plane analyses, the Newthon-Raphson modified, with a step size of 0.02 and a convergence criterion in terms of relative energy variation of 1.0E-03. are adopted for solving the nonlinear equations.
Figure 63 presents the obtained seismic coefficient-displacement curve. Hence a capacity curve, describing the out-of-plane behaviour of the wall when subjected to an incremental loading proportional to the mass, is presented.

![Displacement vs. Seismic Coefficient](image)

Figure 63—Capacity curve of the masonry wall (out-of-plane) subjected to a horizontal loading in the z-direction: Peak value $\alpha = 1.20$, Last converged step $\alpha = 1.03$.

In this case, the out-of-plane behaviour of the model considering all the gravitational loads (self-weight and distributed load on the façade) is studied. As it is possible to observe, the wall is able to sustain an out-of-plane load until a seismic coefficient of $\alpha = 1.20$. This means that the structure is able to resist to a load higher than its own weight in the horizontal direction.

After reaching the maximum value it is possible to observe a post-peak behaviour of the wall. As it was expected for the out-of-plane response of a masonry wall, a brittle behaviour is observed. This consideration is supported by the literature (Babaeidarabadi, 2013). The last converged step has a value of $\alpha = 1.03$. The results of the push over analysis are now evaluated more in depth, describing other important outputs of the numerical analysis.

The deformed shape of the model is shown in Figure 64. The maximum displacement is registered on the mid-span at the top (node 1057). The displacement in z-direction, obtained at the maximum load is about 1,545 mm. The deformed shape clearly shows the location and type of failure mode under out-of-plane loads. The transversal walls act like restraint against the rotations of the extreme part of the façade, while the mid-span area is prone to rotation. This issue will be investigated more in depth in the next sections.
Figure 64–Contour map of the displacement in the horizontal direction at the peak seismic coefficient ($\alpha=1.20$) for the URM out of plane model: Maximum displacement $d_z=+1.545$ mm.

In order to assess the load path, the principal compressive stresses are plotted (Figure 65 - Figure 66). The contour maps allow to understand how the applied force spreads in the wall.

Figure 65 –Contour map of the principal compressive stresses at the peak seismic coefficient ($\alpha=1.20$) for the URM out of plane model in the inner part of the wall (Integration Layer 1): Maximum compressive stress $\sigma_3=-0.724$ MPa.
In the out-of-plane model, the number of integration points along the thickness of the shell elements have been increased to 7. Hence, the principal compressive stresses are plotted in the first (inner) and in the last (outer) layers. In the inward part (Figure 65) of the wall the highest compression values are concentrated in the mid-span, above the opening, and they reach a maximum value of $\sigma_3=-0.724$ MPa, which is only the 7.24% of the compressive strength along the x-axis (10 MPa) and the 9.05% of the compressive strength along the y-axis (8 MPa). On the contrary, on the opposite side of the wall (Figure 66) the compression regards mainly the extreme part of the wall. More specifically the highest values are registered where the façade is restrained: at the base, where the constraint are applied and at the sides were the transversal walls behave like restraints. The maximum values calculated in this layer is about $\sigma_3=-0.325$ MPa (3.25% of the compressive strength of the masonry in the x-direction and 4.06 % in the y-direction). Given the figures of the $\sigma_3$ stresses it is clear that the wall, as expected, has no problem in compression.

In general, for the masonry structures under out-of-plane loads the main problem arises due to the exceedance of the tensile strength. To this extent the principal tensile strains are plotted in order to show the cracking pattern of the model. Also in this case the results are presented in the extreme integration layers. It is clear, observing the contouring plot in Figure 67 and Figure 68 that the situation depicted is exactly the opposite of what has just been described for the compressive regime. In fact, in the first layer (inner part of the façade) the maximum strains are concentrated at the sides of the wall,
while in the outer layer (the outer part of the façade) the maximum strains are mainly located in the mid-span. Moreover, also in the transversal walls a crack pattern is observed, more specifically it is possible to observe on each one of them a concentration of tensile strains where they are connected to the façade. We can conclude then, that not only the façade is interested by a bending behaviour but also the transversal walls are interested by an overturning movement.

![Contour map of cracking pattern at the peak seismic coefficient (α=1,20) for the URM out of plane model in the inner part of the wall (Integration Layer 1). Maximum tensile strain $\varepsilon_1=9.22e-05$](image)

**Figure 67** – Contour map of cracking pattern at the peak seismic coefficient ($\alpha=1.20$) for the URM out of plane model in the inner part of the wall (Integration Layer 1). Maximum tensile strain $\varepsilon_1=9.22e-05$

![Contour map of cracking pattern at the peak seismic coefficient (α=1.62) for the URM out of plane model in the outer part of the wall (Integration Layer 7). Maximum tensile strain $\varepsilon_1=3.07e-04$](image)

**Figure 68** – Contour map of cracking pattern at the peak seismic coefficient ($\alpha=1.62$) for the URM out of plane model in the outer part of the wall (Integration Layer 7). Maximum tensile strain $\varepsilon_1=3.07e-04$
While in the inner layer the tensile cracking limit is not overcome in the outer layer the strains reach a value double of the limits ($\varepsilon_{cr}=1.26e-04$ along the x-axis and $\varepsilon_{cr}=1.77e-04$ along the y-axis, Table 9).

The considerations that have been done for the compressive stresses and for the tensile strains can be summed up in the sketch in Figure 69. The concentration of the tensile and compressive stresses in the external and internal layers are represented. In conclusion, the out-of-plane behaviour of the analysed model seems to be a combination of two of the failure mechanisms described in Paragraph 2.2.2, namely the horizontal bending mechanism and the overturning of the façade (Figure 70).

**Figure 69** – Scheme of the bending behaviour of the model subjected to an out-of-plane loading (top view): localization of the areas where the tensile (+) and compressive (−) stresses are concentrated.

**Figure 70** – Possible out-of-plane mechanism (localization of the plastic hinges): (a) horizontal bending, (b) overturning of the façade.
5.3 TRM-strengthened masonry wall

The model of the out-of-plane wall is now investigated with the addition of two TRM layers, first applied only on the outer side of the wall (1\textsuperscript{st} layout) and then applied on both sides (2\textsuperscript{nd} layout).

As explained in Section 3.4.2, the reinforced wall is modelled using the CQ40L elements in DIANA (in grey shading in Figure 71). They are 8-node layered Quadrilateral shells, in which it is possible to define for each layer a different material. For the masonry and the mortar layers the properties reported in Chapter 3, respectively in Table 1 and in Table 2, are assigned. Moreover, the reinforcement is defined as an equivalent grid to which an eccentricity is given in order to locate it in the mortar layer. The transversal unreinforced walls (in yellow in Figure 71), instead, are modelled with simple 8-node Quadrilateral Shell elements (CQ40S).

5.3.1 Push-over analysis

The performance of the reinforced structure, similarly to the previous models, is studied under the effect of a horizontal loading. A monotonic load proportional to the mass is applied in the out-of-plane direction (z-axis Figure 71). The results are presented in terms of seismic coefficient-displacements curves. The push-over curve is drawn considering the horizontal displacement of the same control point (Figure 62) at the mid span on top of the façade (1057) as in the out-of-plane unreinforced case. The analysis has been performed for only one type of reinforcement (PBO, Table 3 in Chapter 3).
The analysis is performed adopting a Newton-Raphson Regular iteration method with an explicit step size of 0.02 and a convergence criterion applied in terms of relative energy variation of 0.0001.

Figure 72 – Capacity curve of the reinforced masonry wall (out-of-plane) subjected to a horizontal loading in the z-direction in the 1st layout (one side-reinforcement) and in the 2nd layout (double side displacement): Peak value 1st layout $\alpha=1.56$, Peak value 2nd layout $\alpha=1.65$.

In Figure 47, the response curves of the PBO-reinforced out-of-plane model is compared with the behaviour of the unreinforced structure. The analysis concerning the reinforced models showed some convergence problems. The convergence problem can be related to the approaching to the peak load, thus in this cases the results reported are in the last converged step. Nevertheless, from the obtained displacement-seismic coefficient curves, it is possible to infer that both reinforcement layouts allow to improve the out-of-plane responses. This enhancement is showed in terms of seismic coefficient $\alpha$ (Table 16) but also in the type of behaviour, which appears to be more ductile. However, additional investigations should be carried out, in order to have a better understanding of the post-peak behaviour.

As the results in Table 16 show, the reinforcement applied on both sides of the wall produce a slightly higher increase of the seismic coefficient ($\alpha=1.65$ with an increase of about 37.5% from the Unreinforced wall being $\alpha=1.20$).

Table 16 – Push over results: Seismic coefficient for unreinforced and reinforced out-of-plane models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Unreinforced wall</th>
<th>Reinforced wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Seismic coefficient (URM)</td>
<td>$1.20$</td>
<td>$1.56$</td>
</tr>
<tr>
<td>Seismic coefficient at the last converged step (RM)</td>
<td>$1.65$</td>
<td></td>
</tr>
<tr>
<td>Increase of the Seismic coefficient</td>
<td>-</td>
<td>$30%$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$37.5%$</td>
</tr>
</tbody>
</table>
For what it concerns the displacements, the map contouring plot of the 1st layout of the PBO-
reinforced out-of-plane model is presented in Figure 73. For the models reinforced with the PBO-
reinforced mortar arranged according to the 2nd layout the output is qualitatively the same. As it is
possible to observe the deformed shape has not changed from the unreinforced case, but the value of
the maximum displacement is lower. However, in this case the analyses stopped to converge after this
step, hence it is not possible to deduce with absolute certainty if this is the maximum displacement
that the structure can sustain.

Figure 73 – Contour map of the displacement in the horizontal direction at the peak seismic coefficient
(α=1.65) for the 1st Layout of the PBO-Reinforced out-of-plane model: Maximum displacement
dz=+2,019 mm.

The results in terms of stresses in the local x-axis and in the local y-axis direction for the reinforcement
layers are now presented. For the 1st layout reinforcement (textile reinforced mortar applied only on
the outer side of the masonry wall), The σxx stresses are concentrated in the area were the maximum
bending, and in turn the tensile stresses, of the wall occurs (mid span). However, the largest value is
about 35.897 MPa which is far from the tensile strength of the PBO fibres (3400 MPa). The values of
the stresses in the y-direction are even lower (σyy=5,670) and they are maximum around the opening.
It is possible to conclude that the reinforcement is receiving the tensile strains from the masonry, but it
is well far from failing.
Figure 74 – Contour map of the tensile stresses along the x-local axis at the peak seismic coefficient ($\alpha=1.56$) for the 1st layout of PBO-RM (in the reinforcement): Maximum tensile stress $\sigma_{xx}=35,897$ MPa.

Figure 75 – Contour map of the tensile stresses along the y-local axis at the peak seismic coefficient ($\alpha=1.56$) for the 1st layout of PBO-RM (in the reinforcement): Maximum tensile stress $\sigma_{yy}=5,670$ MPa.
Modelling of the behaviour of TRM-strengthened masonry walls

In terms of values the stresses in the PBO grid in the second layout of reinforcement are similar to the ones obtained for the 1st layout. However, it is possible to observe that in this case the inner reinforced mortar is collaborating against the bending behaviour of the wall (Figure 69). In fact, as Figure 76 shows the maximum values of the tensile stresses $\sigma_{xx}$ are concentrated on the mid-span in the outer layer (a) and on the extreme sides of the reinforcement in the inner layer (b). Moreover, in Figure 77 the contouring plot of the tensile stresses $\sigma_{yy}$ are presented. As in the previous case, the grid reinforcement is not failing.

Further investigations, about the modelling of the reinforced out-of-plane model are needed.
6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The present thesis was aimed to study the in-plane and out-of-plane behaviour of TRM (Textile Reinforced Mortar)-strengthened masonry walls. The results contribute to the existing discussion about the use of this innovative strengthening technique in the masonry constructions. Numerical analyses were conducted in order to test the effectiveness of such composite materials in enhancing the seismic response of masonry structures.

The work is organised in two main topics, in-plane and out-of-plane behaviour, hence two models were defined:

1) **In-plane model.** A simple façade, characterized by the presence of an opening, was investigated in the unreinforced and reinforced (on both sides of the wall) conditions. Three different typologies of reinforcement for the TRM, namely PBO, Basalt and Glass, were considered. The FE model adopted 8 nodes curved shell elements for the masonry wall and the mortar matrix and interface elements in between them simulating a perfect bond behaviour. An equivalent grid reinforcement was embedded in the shells representing the mortar (no slip between the fibres and the matrix was allowed). The seismic response of the structures was evaluated by means of push over Analyses under the combined effect of the gravitational loads and horizontal in-plane loads distributed proportional to the mass of the structure. Time history non-linear analysis was further performed using a real accelerogram (L’Aquila Earthquake).

2) **Out-of-plane model.** The geometry of the second model was defined starting from the previous in-plane model, by adding two transversal walls. The modelling in this case was more complex, as the type of interface elements adopted in the previous case were not suitable to study the out-of-plane behaviour. Hence 8 nodes layered shell elements were used here for the reinforced wall (façade). In this way, a different material property (mortar or masonry) was assigned to each layer according to the geometrical details of the structure. Moreover, an embedded grid reinforcement was defined located in the middle of the mortar layer. Two different layouts were defined for the TRM-strengthening: applied to only one side (outer) of the wall and applied on both sides of the wall. Only one typology of fibres (PBO) for the reinforcement was evaluated. The study of the out-of-plane behaviour was carried out performing push over analyses, under the effect of the gravitational loads and a monotonic horizontal out-of-plane loading proportional to the mass.
The numerical results obtained from the unreinforced and reinforced walls were compared in terms of displacements, seismic coefficient-displacement curves, principal stresses and principal strains. The main findings were as follow:

1) The numerical results show that the TRM-strengthening technique largely increase the seismic performance of the in-plane wall in terms of load capacity and displacements. Furthermore, the tensile strains are better distributed in the reinforced wall in comparison to the unreinforced wall. This leads to the conclusion that the reinforced structure dissipates more energy. No differences were observed regarding the type of failure mechanism in the model studied here. In both cases a rocking motion is observed.

2) The out-of-plane studies showed that the behaviour of the unreinforced wall was more brittle than the in-plane wall. The implementation of the reinforcement produces an improvement in the out-of plane behaviour: the maximum seismic coefficient increases and the behaviour appears to be more ductile. However, further studies should be carried out in order to obtain clearer conclusions.

3) The results of the push over and dynamic Analysis suggest that this innovative technique is effective in improving the in-plane shear and out-of-plane bending capacity of the masonry structures.

6.2 Limitations and recommendations

The available studies in literature on modelling TRM-reinforced masonry structures are rather few. Furthermore, the majority of them deal with the in-plane behaviour. On the contrary, additional investigations should be done on numerical investigation of the out-of-plane performance. The strategy proposed in this thesis is suitable to model all the different components of the tensile mortar reinforced structures, but a refinement in terms of available output is needed.

The out-of-plane behaviour of the wall is studied here by performing a static non-linear analysis (Push Over). It is of great interest to study the out-of-plane performance carrying out incremental dynamic analyses, in order to get more information about the seismic behaviour of TRM-strengthened structures.

In the literature, six failure modes for the TRM composite materials have been identified experimentally: (1) debonding with cohesive failure in the substrate, (2) failure at the reinforcement/substrate interface, (3) failure at the textile-to-matrix interface, (4) sliding of the textile within the reinforcement thickness, (5) tensile rupture of the textile in the unbonded portion, (6) tensile rupture of the textile within the mortar matrix. Until now (to the author’s knowledge) none of the analytical model is available for calculation of the component’s capacity in these failure mode. Further studies on this subject for development of the theoretical and analytical formulas are also interesting.
7 REFERENCES

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