SEISMIC RETROFITTING TECHNIQUES BASED ON METALLIC MATERIALS OF RC AND/OR MASONRY BUILDINGS

Doctoral Thesis

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Forewords

This PhD thesis was elaborated during 2003 and 2008 at Department of Steel Structures and Structural Mecanics of "Politehnica" University of Timisoara in the framework of CESCIMS Interdisciplinary Research Platform, supported by two major European Grants, i.e. FP6 EU PROHITECH and RFCS EU STEELRETRO.

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Timişoara, March 2009 Adrian-Ioan Dogariu

To my parents.

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Seismic retrofitting techniques based on metallic materials of RC and/or masonry buildings

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Abstract:

Two innovative strengthening solutions for masonry walls are presented. First one consists in sheeting some steel or aluminium plates either on both sides or on one side of the masonry wall. Metallic plates are fixed either with prestressed steel ties, or using chemical anchors. The second one is derived from the FRP technique, but applies a steel wire mesh bonded with epoxy resin to the masonry wall. Both these techniques are described together with the experimental program carried out at the "Politehnica" University of Timisoara on the aim to validate them.

The Performance Based Evaluation and Retrofitting Design of masonry buildings is addressed. General principles, the acceptability criteria associated to different performance level and intervention strategies are reviewed. The innovative intervention technique is evaluated in terms of strength and ductility performance criteria. In the second part of the paper this technique is applied to retrofitting and historical masonry building.

A building designed according only to geometrical consideration (as typical at the beginning of the XX century) has been evaluated and consolidated applying a strengthening solution based on metallic sheathing. On this, purpose a Performance Based Seismic Evaluation procedure was applied using an equivalent FE model. This model, experimentally and numerical calibrated, to simulate the behaviour of masonry shear walls strengthened with metal sheathing is applied by ABAQUS code, in order to establish the acceptance criteria, performance levels and building performance.

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ABSTRACT

Many of these structures, located in seismic areas built in the past centuries without any reference to the seismic design rules, and affected by age, several modifications or seismic action may be found today in an advanced state of damage with high seismic risk. Given the high seismic risk of historical buildings, it becomes clear that new technological systems are needed, so as to provide solutions not only to specific structural or architectural problems, but also aiming at improving the global performance of the construction, which is intended as a "system". Also, aspects related to the possibilities to remove the retrofitting system are of high interest, if the intervention proves poorly efficient or if better technologies appear.

The first part of the thesis is intended as a state-of-the-art of the main intervention techniques used nowadays, especially for RC frames and masonry walls. Starting from the presented techniques, the thesis tries to underline the major deficiency of the present retrofitting practice. Moreover, an attempt is done to identify the main feautures of a new techniques needed.

In this context, two innovative strengthening solutions for masonry walls are proposed in the framework of the Thesis, techniques that have been investigated in the framework of FP6 EU PROHITECH project. The first technique consists in sheeting some steel or aluminium plates either on both sides or on one side of the masonry wall. Metallic plates (SSP) are fixed either by prestressed steel ties, or by chemical anchors. The second technique is derived from the FRP technique, but it applies a steel wire mesh (SWM) bonded with epoxy resin to the masonry wall. Both techniques are described together with the experimental program carried out at the "Politehnica" University of Timisoara with the aim to validate them. It could be mentioned that these techniques may be appropriate for the application in the case of weak reinforced concrete diaphragms.

In order to validate the two solutions, an experimental program was carried out at CEMSIG Laboratory (director Prof. Dan Dubina) within the Department of Steel Structures and Structural Mechanics and CESMAST Laboratory (director Prof. Valeriu Stoian) in the Department of Civil Engineering, within the "Politehnica" Unviersity of Timisoara . The experimental work included:

- ˜Material tests on steel and aluminium plates, on zinc coated and stainless steel wires and meshes, and on masonry components, i.e. mortar, clay brick units);
- Preliminary tests on 42 small specimens in order to calibrate the connection (connector diameter, prestress level, appropriate epoxy resin for SWM, and most relevant SWM);
- Full scale tests on 22 large specimens, both under monotonic and cyclic loading (in the case of cyclic loading specimens a completely new experimental set-up was built at CEMSIG Laboratory).

The proposed strengthening solutions are an alternative to FRP technology, enabling to obtain a ductile increase of strength, but without increasing the stiffness of the wall, and they have proved their efficiency. It was concluded that metallic sheathing (SP) mainly increases the ductility, while wire meshes (WM) increases the resistance. Both techniques are more efficient when applied on both sides. The prestressed tie connections seem to be more appropriate and the specimens sheeted with aluminium plates have shown a better behaviour than the ones

sheeted with steel. The proposed strengthening systems, all innovative, were confirmed.

There are ilustrated the most important ways to evaluate the bearing capacity of a masonry wall and buildings by presenting the calculation models in a hierarchical order, from the simple analytical models to advanced numerical procedures. All these approaches are presented for virgin masonry and are there are shortly described ideas for the application manner in the case of the retrofitted elements.

The Performance Based Evaluation and Retrofitting Design of masonry buildings is addressed. General principles, the acceptability criteria associated to different performance levels and intervention strategies are reviewed. The innovative intervention technique is evaluated in terms of strength and ductility performance criteria.

A building designed according only to geometrical considerations (as typical at the beginning of the XX century) has been evaluated and consolidated by applying a strengthening solution based on metallic sheathing. For this purpose, a Performance Based Seismic Evaluation procedure was applied using an equivalent FE model. This model, experimentally and numerically calibrated, so as to simulate the behaviour of masonry shear walls strengthened with metal sheathing, is applied by means of the ABAQUS code, in order to establish the acceptance criteria, performance levels and building performance.

The present thesis proposes a "numerical experimentation" procedure for the analyses and it evaluates the behaviour of the masonry structures retrofitted by metallic plates based on performance criteria.

In the first phase of the procedure, there is built a stable and robust FE Model. Afterwards, in the second phase, a parametric simulations have been performed on wall panels of un-reinforced and reinforced masonry, by considering the real mechanical characteristics, in order to obtain acceptance criteria for the retrofitted elements.

In the third phase have been determined equivalent materials that replicate the behaviour of the retrofitted elements for the global analysis and the assessment of the masonry structures. In the last validation phase, the most critical areas of the building must be selected in order to verify the local behaviour.

As a conclusion, the solutions have shown good behaviour and can be successfully applied in the case of masonry walls. The retrofitting solution based on masonry sheathing with metallic plates especially enhances the ultimate displacement and the steel wire mesh improves the resistance of the walls. By a proper calibration when it comes to the connectors spacing, the steel grade and the plate thickness, wires diameter and spacing of the mesh, the solutions can offer a good behaviour of the retrofitted elements and, by rational positioning, an optimal response of the entire building. Disregarding the strength increase, the major benefit of the proposed techniques is the enhancing of the ultimate displacement, thus offering the structural stability and the protection of the inhabitants, by ultimately delaying the collapse of the buildings.

1. INTRODUCTION

1.1. INTRODUCTORY REMARKS

One of the most important and actual tasks of the civil engineering community is to recover and conserve the existing constructions. The retrofit of historical constructions represents a big problem in earthquake prone areas such as the Mediterranean and Balkan countries, including Romania, which characteristically possess the highest concentration of such historical constructions and monuments.

Many of these structures, located in seismic areas, built in the past centuries without any reference to the seismic design rules, and affected by age, several modifications or seismic action may be found today in an advanced state of damage presenting high seismic risk.

The latest seismic events (Friuli-Italy, 1976; Vrancea-Romania, 1977; Campania and Basilicata-Italy, 1980; Spitak-Armenia, 1988; Banat-Romania, 1991; Erzincam-Turkey, 1992; Dniar-Turkey, 1995; Umbria-Italy, 1997; Adana-Turkey, 1998; Iznit and Duzce-Turkey, 1999; Athens-Greece, 1999) showed that the degree of seismic protection in these parts of Europe is largely unsatisfactory. This evidence showed that the old masonry and reinforced concrete structures (typically after II World War) seems to be the most vulnerable to seismic action; these structures are characterized by low quality of construction and are built without any respect to seismic detailing rules or seismic conformation, by using poor materials and often poor workmanship [1].Given the fact that such buildings can also be the seat of strategic or public offices, civil protection offices, hospitals, common halls, schools and so on, it is easy to understand the gravity of the situation, dramatically testified every time an earthquake occurs [2].

Given the high seismic risk of historical buildings, it becomes clear that new technological systems are needed, so that to provide solutions not only to specific structural or architectural problems, but also aiming at improving the global performance of the construction, which is intended as a “system”. Also, aspects related to the possibilities to remove the retrofitting system are of high interest, if the intervention proves poorly efficient or if better technologies appear. Within the technical field of seismic rehabilitation, two aspects enjoy increasing attention from engineers and researchers, namely [1]:

- The preservation of the Structural Integrity of existing buildings under severe or exceptional seismic actions (SI);
- The improvement of building seismic performance by means of Reversible Mixed Technologies (RMT).

The decision to retrofit a building is mainly related to achieving a satisfying level of Structural Integrity and Safety under severe earthquake actions. The application of Reversible Mixed Technologies is, in some cases, the only tool in order to achieve a satisfactory level of SI. The concept of Structural Integrity relies on the necessity to ensure seismic protection against collapse also in the case of destroying events. In this view, it can be properly framed within the advanced concept of Performance Based Design (PBD). Until now, the Performance Based Design has been only applied to new structures, which can be easily designed by complying with relevant behavioural thresholds set by PBD itself. There are no direct applications in the field of existing constructions. In particular, neither criteria nor methodologies are available for achieving a satisfying design level against strong intensity earthquakes. This is indirectly confirmed by most national seismic codifications, which, as a matter of fact, allow avoiding a rigorous seismic retrofit in case of historical constructions. This approach surely tends to preserve the monumental value of the construction, but doesn't really protect it against severe earthquakes. This aspect obviously deserves great attention not only in the perspective of saving human lives, but also in the light of preserving invaluable artefacts from complete destruction. The use of innovative materials and Mixed Technologies is the most appropriate answer to ensuring an adequate performance, and hence the Structural Integrity, under strong seismic actions [1].

Reversible Mixed Technologies (RMT) are based on the integration of structural members of different materials and/or construction methods into a single construction. The basic feature of RMT is that their application should always be completely recoverable, that is reversible, if required. This is considered as an essential design requirement in order to protect historical and monumental buildings against unsuitable rehabilitation operations. The main aim of RMT is the best exploitation of material and technology features, in order to optimize the structural behaviour under any condition. This practice, initially concerned with new, technologically advanced buildings, is now being looked up with increasing interest in the field of structural rehabilitation, too, due to the greatest possibilities of structural optimization and, hence, performance maximization, achieved thanks to mixed technologies. In a few words, the use of reversible mixed technologies would lead to the best exploitation of each material and/or technology used in the intervention, providing in such a way the best performance both from the technical and economical point of view [1].

The future will surely focus more on recovering and maintaining the old buildings which need structural consolidation and functional rehabilitation. Nowadays, a lot of resources are spent in order to retrofit existing buildings, by improving their overall structural and in particular seismic safety, nevertheless designers must deal with a large variety of problems without any well defined methodology at their disposal.

The importance and value of the building is also a very significant issue if the decision to retrofit is made. When historical buildings are concerned, there needs to be operated a delicate selection of the consolidation materials and technologies. A clear distinction can be made between the new materials which represent the "medicine" and the old ones which are the "sick". As a "medicine" can use both the traditional materials, like cement, mortar, reinforced concrete and steel, and the innovative materials, like special mortars, polymeric and composite material, special metals (high strength steels, stainless steel, aluminium alloys, titanium alloys, etc.) as well as some special devices belonging to advanced systems of seismic protection by means of passive control technologies [160].

The first problem that must be solved is to choose the appropriate material and technique. Moreover steel possibilities in this field are mainly unknown in common practice, and steel is used only in very particular cases or by a few skilful technicians. Previous research and studies were actually limited to the analysis of very specific technical/scientific problems without any reference to common problems for direct use in retrofitting design practice. It is obvious that in a similar situation, the solutions proposed by designers are often not well optimized from the economical, structural and constructional point of view. It is important to find designers with a complete set of tools for the use of steel solutions in seismic retrofit of existing reinforcing concrete and masonry buildings with reference to problems currently found by engineers and other operators from the construction sector. Many current projects allow the increase of the use of steel in seismic retrofit of existing buildings, thus improving the safety and the quality of constructions and reducing the price of interventions as well as increasing the possibilities of reversibility and eventual post-earthquake repairing of new interventions and their degree of prefabrication [2]. This thesis dwells on masonry and reinforced concrete retrofitting solutions based on using metallic materials and modern and innovative techniques.

Nowadays the retrofitting practice in Romania is still based on traditional techniques that use affine materials. Some steps to align the Romanian regulations to European practice have been made, by the issuing of P100-3 [171] that introduces for the first time the Performance Based Seismic Assessment concepts. Even so, the Romanian administration in charge with cultural heritage preservation imposes strict conditions in the case of structural retrofitting. Metallic based techniques have become more and more attractive, due to their reversible character and of course high mechanical properties. There is high interest in Romania to develop retrofitting technologies that should use metallic elements, from the technical and design point of view.

The thesis aims to presenting and to evaluating performance based criteria and at validating an innovative retrofitting on masonry walls or weakly reinforced concrete diaphragm solution based on sheathing the shear walls with metal plates, mild carbon steel or aluminium alloy, connected through chemical anchors or prestressed ties.

These introductory considerations are fully based on the PROHITECH and the STEELRETRO rehabilitation philosophy, strategies and problematic stated in their project proposal.

1.2. SUMMARY OF THESIS

This chapter presents the contents of the thesis and the European research context in which the research program of this study was carried-out. The importance and high interest of retrofitting existing building by using reversible mixed technologies is emphasized.

Chapter 2 makes a state-of-art of traditional and modern retrofitting techniques emphasizing the advantage of the use of metallic materials. General issues of structural intervention starting from the mechanical causes of damage, the logical phases of intervention, the technical and administrative steps and different levels of consolidation are shortly overviewed. The main characteristics of the new strategies for anti-seismic protection of buildings are presented. As final remarks, there are presented the advantages and disadvantages of current retrofitting techniques and two major desiderates of the current retrofitting techniques practice i.e. reversibility and mixed character. More attentions is given to retrofitting techniques applied to masonry walls, underlining the advantages and disadvantages, trying to define the actual context and needs in this field and the advantage of the proposed retrofitting techniques.

Chapter 3 presents the concept of Performance Based Seismic Assessment and Design in case of existing buildings, manly based on the American regulations as FEMA 356 and European Codes EN1998-1-3. Basic principles are overviewed concerning the strategy, the concept and the details. In this chapter there are defined the rehabilitation objective, the performance levels and the acceptance criteria by underlining aspects regarding the choice of analysis method and intervention strategy. Some provisions available in American codes are also mentioned. A decisional matrix is proposed for the evaluation and selection of a technique, based on structural, technical and economical aspects.

Chapter 4 is dedicated to the experimental work carried-out within the framework of thesis research program. An innovative retrofitting technique of masonry shear walls based on metal sheathing with mild carbon steel or aluminium connected through chemical anchors or prestressed ties is validated experimentally. The experimental program include material tests on masonry components (i.e. brick units and cement mortar), on steel and aluminium plates, connectors and wires and wire meshes. Calibration tests on 500 x 500 x 250 masonry specimens have attempted to clarify the behaviour of the composite steel–masonry system and the application details.

Tests on 1500 x 1500 x 250 mm masonry panels retrofitted with the proposed retrofitting solution have been performed under monotonic and cyclic load conditions. For the cyclic tests a new testing frame was designed and built. A simplified method was used for a synthetic presentation of the results. The tests have especially emphasised the important benefit of the technique in the range of life safety – collapse prevention performance levels range.

Chapter 5 presents the modeling strategy of unreinforced masonry panels from simple strut models to more complex finite element approaches. The main approaches are listed that treat the homogeneous material as a continuum material, and some possibilities to model discontinuous nature of the masonry. The principles of limit analysis are stated. Where the approach is appropriate for the proposed retrofitting techniques, possibilities are suggested on how to apply these calculation models.

A complex numerical model of the retrofitted solution is built, that takes into account the masonry panel, steel plates and connectors' behaviour, by using ABAQUS code and in order to simulate the positive effect of the technique and to establish clear performance criteria.

Chapter 6 tries to offer a tool for the design of the proposed retrofitting technique. A first proposal is related to an experimental based design. Basically this procedure experimentally establishes a unitary capacity of the wall, by comparing the capacity of the entire wall from a direction from a certain level with the storey seismic force, the safety level of the building is assessed. A more complex tool, with the use of advanced numerical models is applied on a real masonry building. Global analysis are performed and performance objective of the building is checked, by using performance based criteria, in terms of plastic strain, and an equivalent material, able to replicate the real behaviour of the mixed system masonry-steel. Some simple analytical formulas may be proposed treating retrofitted masonry panel as reinforced masonry.

Chapter 7 underlines the main contribution of the thesis and draws final conclusions regarding the aspects presented in this thesis. A list is also drawn on the papers, conference and grants related to the thesis topic in which the author has been involved.

1.3. INTERNATIONAL RESEARCH FRAMEWORK

The innovative retrofitting solution was proposed and validate, experimentally and numerically, within the framework of PROHITECH (FP6 INCO-CT-2004-509119/2004 Earthquake Protection of Historical Buildings by Reversible

Mixed Technologies), developed under the coordination of Prof. F.M. Mazzolani, in which "Politehnica" University was involved as a partner. The research program involves 16 academic institutions coming from 12 Countries, most of them from the South European and Mediterranean area.

The main scope of the research project PROHITECH was to develop sustainable methodologies for the use of reversible mixed technologies in the seismic protection of existing constructions, with particular emphasis on historical and monumental buildings. This would primarily involve saving human lives and reducing both economic and cultural losses caused by earthquakes. Reversible mixed technologies exploit the peculiarities of innovative materials and special devices, allowing ease of removal if necessary. At the same time, the combined use of different materials and techniques yields an optimization of the global behaviour under seismic actions [1].

The main subject of the research was represented by relevant buildings erected from the ancient age to the first half of the 20th century, which can be considered as belonging to the cultural heritage of the involved Countries. Historical constructions cover a wide and diversified range of structural categories, including masonry, steel and reinforced concrete buildings, needing to be fitted with adequate seismic resistant provisions. The proposed activity is mostly focused on the use of innovative technologies, namely those relying upon mixed reversible systems. Consequently, a more advanced understanding of material and device behaviour, as well as a deeper insight into the seismic response of constructions is required.

The project proposed the following objectives [1]:

- Drawing the attention of industry, research centres, engineers and competent authorities of European and Mediterranean Countries on the problem of safeguard of the construction heritage from seismic risk, in particular when historical buildings are concerned;
- Improving the awareness of the operators listed above about the importance of using advanced materials and technologies in the seismic up-grading of constructions;
- Improving the average knowledge of practicing engineers about innovative systems of seismic protection, so as to contribute to the institution of specialized skills in the field of seismic rehabilitation;
- Promoting the use at a wide scale of reversible and environmentally friendly technologies, in order to fit existing constructions with easily removable and modifiable seismic protection systems;
- Supporting the adoption of "smart" materials and special techniques for the seismic protection of constructions as a cheap and effective alternative to traditional, highly intrusive strengthening methodologies, especially when historical constructions are faced;
- Advancing the state-of-the-art in the field of seismic protection of constructions, by adding new information about the behaviour of structures fitted with special systems and/or using advanced materials or devices for improving the seismic performance;
- Allowing engineers to use simple and reliable tools for analyzing the behaviour of constructions provided with advanced systems for seismic protection, as well as for detailing up-grading interventions;
- Developing advanced, Performance Based Design (PBD)-complying guidelines for the practical application of innovative materials and technologies in the field of seismic restoration.

In order to achieve the objectives listed above, the work plan of the research program was divided into 12 scientific Work Packages as follow [1].

- Overview of existing techniques;
- Damage assessment;
- Risk analysis;
- Intervention strategies;
- Innovative materials and techniques;
- Reversible mixed technologies;
- Experimental analysis;
- Numerical analysis;
- Calculation models;
- Validation of innovative solutions and procedures;
- Case studies;
- Design guidelines

The Performance Based Seismic Assessment with Decisional Matrix were proposed and detailed in the frame of STEELRETRO (RFSR-CT-2007-00050 – Steel Solutions for Seismic Retrofit and Upgrade of Existing Constructions) coordinated by RIVA Acciaio S.p.A. In this RFCS projects are involved 11 universities, research centres and industrial companies from 7 countries. Also the benchmark proposed in this project was taken as study case of this thesis. The aim of the STEELRETRO research proposal is to set up steel solutions for the seismic retrofit of existing buildings, furnishing design and construction methodologies, tools for dimensioning of elements and connections as well as for cost estimation [2].

The project was divided in 9 Working Packages [2]:

- Analysis of the main problems affecting existing buildings in seismic areas;
- Development of a performance based design methodology for existing buildings and its application to retrofitting or upgrading systems actually used in European seismic Countries;
- Cost, performance and construction analysis of steel solutions to retrofit or upgrade vertical systems;
- Cost, performance and constructive analysis of steel solutions to retrofit or upgrade floor systems;
- Cost, performance and constructive analysis of steel solutions to retrofit or upgrade roofing systems;
- Cost, performance and construction analysis of steel solutions to retrofit or upgrade existing foundations;
- Experimental analysis on proposed structural details or connections and dissipative systems;
- Application to case studies;
- Design and construction rules.

The main results of the research program were presented in COST 26 Action - Urban Habitat Constructions under Catastrophic Events (Chairman Prof. F.M. Mazzolani) – WG2 – Earthquake resistance, chaired by Prof. Dan Dubina and Prof. Alberto Mandara. This COST deals with the outstanding topic of the protection of constructions in urban areas from exceptional loads, such as earthquakes, fire, wind, impact, explosions and so on. Buildings in urban habitat are designed, in fact, according to rules aimed at ensuring an adequate structural safety level under "normal" loading conditions. Nevertheless, all structures can be exposed to certain extreme conditions arising out of predictable natural or man-made hazards. These include earthquakes in non-seismic areas, unforeseen fire, exceptional wind storms, heavy snow loading, gas explosions, accidental and/or incidental impact from projectiles or vehicles out of control, and explosions due to bomb blasts during terrorist attacks. The present Action aims to establish towards an improved understanding of the response of constructions to such extreme conditions, in order to ensure a given adequate safety level.

The obtained results have also been disseminated within the framework of bilateral projects of "Politehnica" University of Timisoara with Aristotle University of Thessalonica, sustained by the Romanian Research Ministry, as "Conservation and rehabilitation of historical buildings using light gauge metal (director Prof. D. Dubina 2004-2005) and "Strengthening and rehabilitation of historical buildings by reversible technologies" (director assoc. Prof. Aurel Stratan 2006-2007).

All these European research grants and many other show the actual interest and preoccupation of the civil engineering scientific community and European authorities within the topic of Metallic Based Anti-seismic Retrofitting Techniques, the main subject of the present PhD. Thesis.

2. TRADITIONAL AND MODERN RETROFITTING TECHNIQUES

2.1. METAL BASED CONSOLIDATION TECHNIQUES OF EXISTING BUILDINGS

2.1.1. Advantages of metallic materials

Because of the wide range of steel types that can be found on the market, hot rolled or cold rolled profiles, steel sheeting, tube sections, I, H sections etc., and of the wide range of mechanical characteristics that it can possess, steel can be used with a high degree of flexibility and is able to solve almost any consolidation issue.

The possibilities are thus numerous and can be extended to a wide range of operations from a simple intervention of consolidating a single structural element to restoring the whole structural assembly taking into account the antiseismic behaviour of the building.

In seismic areas which are present all over the national territory of Romania, the problem of static rehabilitation of buildings becomes an increasingly delicate matter because of the need to offer either new or existing buildings sufficient strength in the case of seismic motion. In the same time, the problem of fast and efficient rehabilitation of buildings affected by earthquakes arises in the process of facilitating the social-urban rebuilding of the affected zones. From this point of view steel products fully satisfy the need to create in a short amount of time provisional buildings which can be easily removed, thus making room for the final consolidation of the buildings.

The examples in this paper show what can be achieved by using steel or metallic products for rehabilitation, the concept of the method, the theoretical base and the possibility of further development.

An optimal solution to accommodate these needs is the use of steel structures that have a modern and reversible character.

Steel has clear advantages given by his structural characteristics, like [135]:

- clear shape definition,
- prefabrication,
- reversibility,
- high mechanical strength,
- mechanical isotropy,
- reduced dimension and weight,
- ease of transportation,
- ease of application,
- ease of manoeuvring in tight spaces,
- workability,
- commercial availability,
- prefabricated steel of different shapes, dimensions, mechanical characteristics,
- recyclable.

Details on these characteristics [135]:

Prefabrication – a major part of the labour is done in plants, thus reducing the on-site labour to an assembly process. This leads to the optimization of the productivity and making on-site labour in narrow spaces easier. Steel elements can be prefabricated at optimal dimensions, thus facilitating transport and manoeuvrability. The severe tolerances imposed are easily respected by prefabrication in plant and the on-site assembly done with bolted connections becomes very easy.

Reversibility is a characteristic of steel present mainly in the case of bolted connection. This advantage can be exploited in different types of interventions, like: provisional works for protection, bearings, made from tubular systems connected by mechanical fasteners for passageways etc.

The reduced weight of elements is of great importance especially when the new structure interacts with the old structure. The reduced weight in relation with high strength minimize the supplemental load of the existing walls of the old structure, which is very likely to be deteriorated or severely compromised by modifications during their lifetime by atmospheric agents or by earthquakes. Reduced weight also implies fewer problems with transportation and manoeuvring, the different elements can be set in place on site using light machinery or even manually.

Reduced structural dimensions help in the preservation of the existing building when the static function of it was eliminated or integrated into the new structure. That is why in most cases it is recommended that the new structure be left visible.

Ease of application makes steel an ideal instrument in all cases especially when there is a short time available for execution of the salvage or of interventions. This quality is most useful in emergency cases that follow a calamity and require bracing.

Great commercial variety of products offers flexibility in meeting any design requirements or execution demands. Steel products can be found as hot rolled profiles (I, T, L, U etc.), cold rolled profiles, pipes, tubes, profiled sheeting, plates, strips etc. Steel offers strength and ductility and can be used in any procedure to replace old damaged elements, which are no longer capable of carrying out their static function, and in any procedure to integrate new elements in the existing structures and in complete reconstruction procedures.

All the above presented qualities makes steel an optimum material for all types of consolidation works of different elements and assemblies, especially those in seismic areas.

2.1.2. Modern metallic materials used in consolidation

Modern metallic materials have not yet found an ample application in the field of civil engineering and especially in the consolidation domain. The lack of application on a large scale of these products can be justified on one hand by the lack of clear theoretical reference regarding their use in the consolidation field and on the other hand by the lack of a code that validates the use of these materials in structural applications. Despite having a high cost compared to regular steel materials such as stainless steel, copper, aluminum alloys, titanium alloys or shape memory alloys, they have a whole range of advantages due to their characteristics [152]. A comparison of their characteristics with respect to steel is presented in the Table 2-1:

Table 2-1 Main characteristic of metallic materials [143]

Material	γ (g/cm^3)	E (kN/mm^2)	$f_{0,2}$ (N/mm^2)	f_t (N/mm^2)	ε_t x100 (A_5)	α x 10^6 ($^oC^{-1}$)
Mild carbon steel	7.85	210	235÷365	360÷690	10÷28	12÷15
Low yield steel	7.85	200	90÷100	300÷350	40÷60	12÷15
Stainless steel	7.8	196	200÷650	400÷1000	10÷40	17÷19
Aluminum alloys	2.7	65÷73	20÷360	50÷410	2÷30	24÷25
Titanium alloys	4.5	106	200÷975	300÷1100	8÷30	6÷7
Copper	8.2÷8.9	88÷118	70÷400	170÷720	6÷50	18
SMA Ni-Ti	6.5	28÷75	100÷560	750÷960	15.5	6.6÷11

The advantages of these materials can be summarized as follows:

- high corrosion resistance;
- high weight-strength ratio;
- good ductility;
- ease of application and manufacture;
- aesthetics appearance;
- reversibility and availability on the market.

A great advantage of these materials (except for low yield steel) compared to normal steel is the high corrosion resistance which makes it possible for these materials to be used in aggressive, high humidity environments leading to reduced maintenance costs. The choice of whether to use these materials is not only done by taking into account mechanical characteristics like strength, deformation capacity or ductility but rather from a more general point of view regarding their physical and chemical compatibility with existing materials, resistance to corrosion or the possibility to obtain particular shapes with aesthetic value.

For these materials as opposed to normal steel whose mechanic behaviour is reduced to a simple elastic-plastic model, their behaviour curve σ–ε is a continuous one without yielding branch. It is necessary to adopt a more complex nonlinear behaviour which leads to the conclusion that these materials do not always exhibit the same ductility as soft steel and the evaluation of their behaviour at ultimate limit state must be done by a strength calculus at a certain state of deformation.

2.1.2.1. Stainless steel

Stainless steel is obtained by adding chrome, nickel and nitrogen to weak alloyed carbon steel forming an invisible protective layer (Cr_2O_3). Added together there are over 60 types of stainless steel classified as: martensitic, feritice, austenitic, duplex or hardened by mixing, depending on their microstructure. The most common alloy is that of austenite category and is based on adding chrome and nickel, which offer the best chemical resistance in relation to their consolidation capacity. By using stainless steel in the field of consolidation we have the possibility to hide the strengthening elements, without being at risk of reduced performance during its lifetime. This proves to be very useful in the case of statues, columns or other stone elements for which any surface systems would be incompatible with the aesthetic aspect of the monument.

An example of using stainless steel is the consolidation of the main hall of the „Mercati Traianei” (Emperor Traian Square) in Rome, which was severely damaged after some modifications made in the past. The consolidation works mainly focused on obtaining a stainless steel confinement system starting from a mechanical model of masonry confinement [142].

Stainless steel can also be used successfully for oleo dynamic dissipation devices conceived to improve the seismic performance of buildings. Using such a material for sliding devices considerably lowers the maintenance cost.

2.1.2.2. Aluminum alloys

The strong point of this material lies in its reduced weight (approximately one third of that of structural steel) and its good resistance to corrosion, leading to a minimal increase in mass and in the same time reducing maintenance problems. The alloys used are obtained by adding pure aluminum, which has low strength but good ductility, magnesium elements, silicone magnesium, copper, zinc, manganese etc. Thus the mechanical characteristics obtained in this manner are various. By using aluminium alloys for structural applications, there can be obtained great freedom and a wide range of intervention solutions. Aluminum was used mainly in the addition of storeys and intermediate light storeys. In both applications the reduced weight and corrosion resistance characteristics are used to a maximum. These two properties combined with aesthetics and the possibility of easy replacement led to the choosing of aluminum as a material for the ribbed structure at „Emperor Traian Square” in Rome, which protects the existing ruins and creates new spaces for public access. The new structure based on GEO-systems was intended to have both reduced weight and high structural performance but also to offer a simple and efficient system to construct [158].

Aluminum alloys were also used in the case of the modification of the sheeting of suspended bridges in France [145], at Trevaux, Montmerle, Grosolee, near Lyon, to reduce dead load on the bridge piles [161].

Due to a the reduced elastic limits (7 times), although the elastic modulus is also smaller (3 times) in comparison to mild steel, aluminium enters in the post-elastic range and starts dissipating energy at a smaller elongation (~2 times). For this reason, it can be suitable for used in the case of brittle materials consolidation. Even if the ultimate elongation is smaller than for mild steel, in case of brittle materials, a large deformation is not required, because this deformation cannot be accommodated anyway.

2.1.2.3. Titanium alloys

Titanium alloys are obtained by mixing molybdenum, vanadium, aluminum and steel. They are divided into three categories:

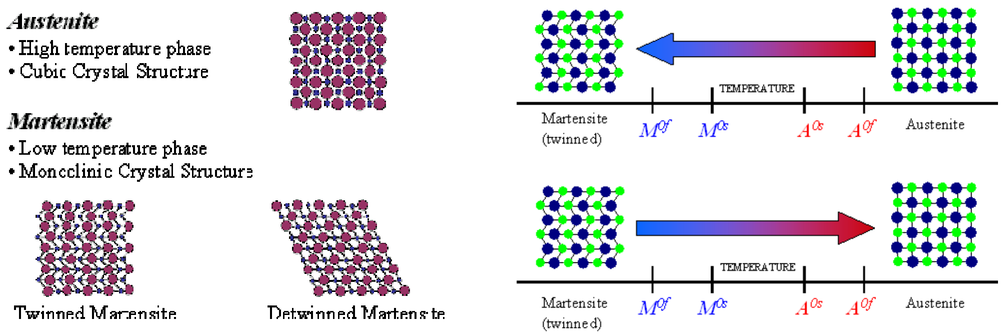
- Alpha alloys – no thermal treatment, medium to good strength and ductility;
- Alpha-beta alloys – with thermal treatment, medium to good strength, but without having the same behaviour at high temperatures as the alpha;
- Beta and non-beta alloys – light thermal treatment, with good strength and good behaviour to medium temperatures.

All titanium alloys have very good corrosion resistance and can be used in the extrusion process. The most important property in the field of consolidation is to have a small thermal expansion coefficient, very close to volcanic or metamorphic rocks like granite or marble. This fact enables for marble elements the use of efficient systems that do not induce an additional state of tension and assure where needed a state of pretension, independent of temperature changes [143]. Titanium based techniques were successfully used in the case of consolidation of the

Parthenon in Athens or Anthony’s Column in Rome [84], where they proved much more useful than steel based solutions used before, which had induced a series of cracks in the existing structure due to thermal expansion and corrosion.

2.1.2.4. Shape memory alloys

Shape memory alloys (SMA), most Ni-Ti or Cu-Al-Zn, can be regarded as „intelligent” materials, because both the yield limit and the elastic modulus increase as the temperature rises up to the transformation temperature. This temperature corresponds to a solid transformation between the martensitic phase and austenitic state (see Figure 2-1).



(a) (b)

Figure 2-1 SMA states (a) and transformation of SMA with temperature

This behaviour is limited by the transformation temperatures M_f and A_f, temperatures. These temperatures correspond to the limits between which only martensite and austenite structures can exist. The above transformation can be induced either by mechanical means or by changes in temperature, thus obtaining the initial capacity and state of deformation either spontaneously or by heating.

In the first case (mechanical measures), the behaviour is called super elastic (Figure 2-2) and leads to the complete disappearance of deformations following the unloading. Because of the different steps of loading-unloading, an important amount of energy is dissipated during this cycle.

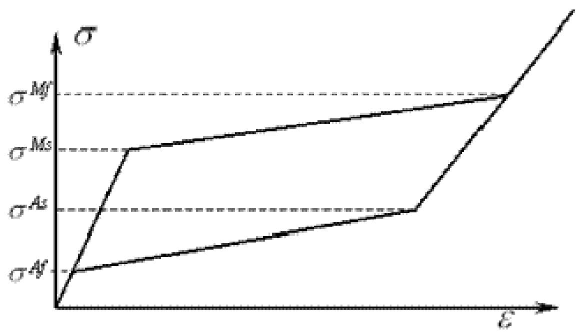


Figure 2-2 Super elastic behaviour

In the second case a full recovery of the material is obtained when the material is heated to a temperature above step A_f. Such behaviour is explained by the memory effect (see Figure 2-3).

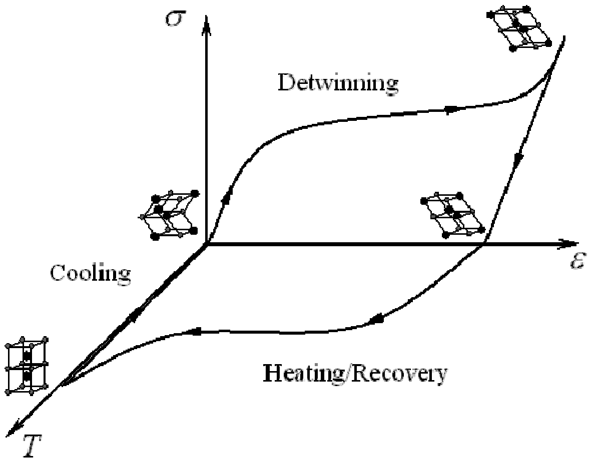


Figure 2-3 σ–ε–T behaviour due to memory effect

When the material is prevented from deforming freely, very high internal stresses can be reached due to the high rigidity of the material in the austenic phase. In the field of structural consolidation the memory effect can be used when it is desired to introduce an initial state of tension in an element. This can be applied in the case of confining elements, especially for masonry elements. SMA are fitted at temperatures lower then M_f and then heated to reach the temperature A_f, so as to obtain the wanted state of tension.

The super elastic character can be used with good results in designing anti-seismic devices due to the high energy dissipation during one complete cycle. In the field of seismic protection of monumental buildings, the first application took place in Italy, in the consolidation of St. Francis Church from Assisi – Italy, which had suffered from severe damage during the 1997 earthquake [162]. The SMA devices were placed between the tympanum and the roof of the church together with oleo-dynamic devices in order to increase energy dissipation. Similarly, such devices combined with vertical pretension rods were installed in the case of the St. George Church din Trignano (Italy), resulting in an increase in bending capacity and energy dissipation [162].

2.1.2.5. Copper alloys

These alloys have been used since Antiquity in civil engineering and are nowadays mainly used as electric conductors. These alloys have interesting perspectives in the field of the rehabilitation of building heritage due to the wide range of dimensions and shapes and mechanical characteristics. In combination with zinc or bronze, yellow alloys can be obtained. These alloys have good corrosion resistance which makes them useful in the case of protection treatments [143]. Also they pose high ductility due to their chemical composition which helps in the process of rolling into thin sheets. The use of copper and its alloys in the field of consolidation is advisable, firstly because of their aesthetics. Their good workability helps obtain different shapes and facilitates the recovery and recycling of the excess material. There is also the possibility to have different textures and colours and they can also be used with a structural role due to their mechanical characteristics.

It is worth mentioning that in Northern Europe copper is frequently used in civil engineering especially for roof covering.

2.2. GENERAL APPROACH OF STRUCTURAL INTERVENTION

2.2.1. Consolidation levels

The general criteria that govern the structural consolidation are linked with the preliminary definition of the level of intervention. Pinpointing the most appropriate level of intervention depends on multiple variables concerning: the importance of the building, the destination of the building after consolidation, the technological systems engaged, the degree of safety necessary, and last but not least the available funds.

A classification of the level of importance can be put in relation with the fundamental stages of the logical and chronological process which has the final purpose of recovering of the static efficiency of the building and its preservation in time.

The levels of intervention are [133]:

- Safeguard;
- Repair;
- Reinforcing;
- Restructuring.

SAFEGUARD implies a set of measures meant to assure the safety of the building in the transitory phase, if important damage occurs after an earthquake.

REPAIR implies measures that aim to restoring the initial structural efficiency, prior to earthquake action.

REINFORCING and RESTRUCTURING, do not imply, as opposed to the first two levels, the existence of damage that compromises the safety of the building, but are necessary when functional reorientation of the building and its alignment with new standards of strength and stability are desired.

2.2.1.1. Safeguard

Safeguard represents the first level of intervention that consists of provisional measures meant to assure an adequate level of safety in the transition phase. This precedes any subsequent interventions that have a definitive character.

It refers to the situations where quick recovery interventions are needed until partial or total consolidation of the building is done. It is used both for the protection of the site and the preventions of collapse.

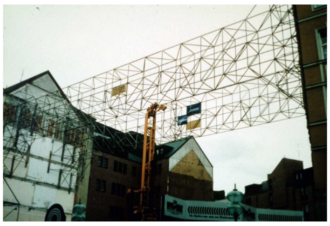
Typical cases when safeguard measures are needed are the emergency cases that follow an earthquake, when the lack of material requires a quick and simple intervention with maximum efficiency.

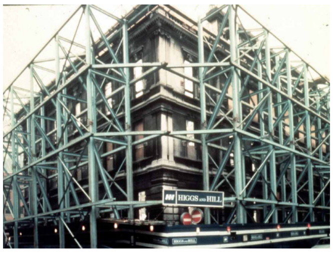
For these cases steel elements are recommended because of the ease of application, their prefabricated characteristic, ease in transportation and assemblage regardless of the atmospheric conditions and last but not least the economical advantage that derives from the recycling and possibility or re-using steel.

Field of application

This intervention level finds its utility in the case of:

- temporary support of a building during the reconstruction of a new building placed between two other buildings (Figure 2-4 a);
- steel structures that support the façade of a building during the demolition of a part of this building for replacement (Figure 2-4 b);
- temporary support for building façades in the consolidation phase after a earthquake (Figure 2-4 c);
- provisional roofing during consolidation works (Figure 2-4 d & e) [148].


(a)


(b)


(c)


(d)


(e)

Figure 2-4 Safeguard application (Mazzolani) [159], by courtesy of F.M. Mazzolani

Technical details of the intervention

Tubular steel scaffoldings are used as “ad hoc” solutions that present good flexibility and optimal results.

Heavy steel structures (welded sections or rolled sections with bolted connections) and light steel structures (bolted tube sections) proved very effective in passive or active safeguard measures.

The character of the safeguard measures must satisfy the following conditions [133]:

- speed of execution;
- flexibility of the constructive systems;
- adaptability to narrow and hardly accessible work spaces;
- reversibility of the intervention;

In this phase traditional materials as masonry and especially timber still represent a viable alternative.

Provisional works for support and protection have a temporary character and belong to the framework of safeguard interventions. Provisional works are used even during the execution of final consolidation works (to protect the building during replacement of the covering), or during the erection on a new building (scaffolding, passageways).

Until a few years ago this type of works has been done with timber elements, for example passageways and support struts etc.

In the framework of modern solutions, prefabricated steel, due to the current technology can be easily implemented „ad-hoc" due to its great flexibility and diversity of shapes and sizes. Their advantages consist in the ease of removal and possibility of re-using. Thanks to the great number of possibilities offered by the constructive system, tubular scaffolding can be used in any typical consolidation works. Reticular structures with braces can be built as real supporting structures – scaffolding with excellent set up and dismantling time, offering a viable alternative to brick masonry „contraforts" type structures.

One must not forget about works done as a precaution, capable to prevent landslides or any other phenomena, not entirely controlled, that tend to cause degradation of the existing structure.

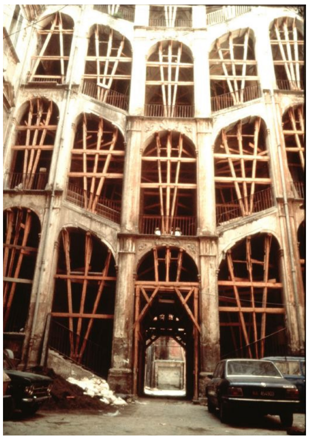
Examples of application

In the field of safeguard, passive interventions, structural masonry elements are used as contraforts to support the façades of buildings, especially in Mediterranean countries (Figure 2-5). This system has the disadvantage of high weight of the masonry supports that require their own foundation, and in many cases the support structure fails before the consolidated building.

(a) (b)

Figure 2-5 Contraforts application (Mazzolani) [159] by courtesy of F.M. Mazzolani

Timber struts of different types and shapes are inserted into the masonry structures in order to achieve an even distribution of vertical loads before the consolidation works (Figure 2-6 a) or to assure lateral support and access facilities (Figure 2-6 b). Bamboo structures and scaffoldings are widely spread in Asia as service systems during refurbishment operations. (Figure 2-6 c). There is the advantage of the reduced weight with satisfactory strength, ease of manipulation and good workability, flexibility and adaptability of the material in all situations, and the reduced price makes of timber a very frequent choice in this field.


(a)


(b)


(c)

Figure 2-6 Wooden safeguard works (Mazzolani) [159] by courtesy of F.M. Mazzolani

An interesting solution proposed for supporting the Pisa tower in order to avoid collapse is the use of a steel ring which surrounds the tower as presented in Figure 2-6.

Steel elements both for passive intervention and active ones proved efficient in many cases.

Active support during the restoration of the York Cathedral (Figure 1.8). The lateral façade of the building, deviated 635 mm from the vertical, was temporarily supported so as to allow the consolidation of the foundations. The intervention consisted in the use of a steel reticulated structure elevated on independent concrete foundations in order to sustain a couple of hydraulic jacks which introduce a constant pressure to compensate for the effect of thermal variation. The jack system guaranteed for the introduction of a constant load in the masonry structure during the under-foundation works. [49].


a)

b)

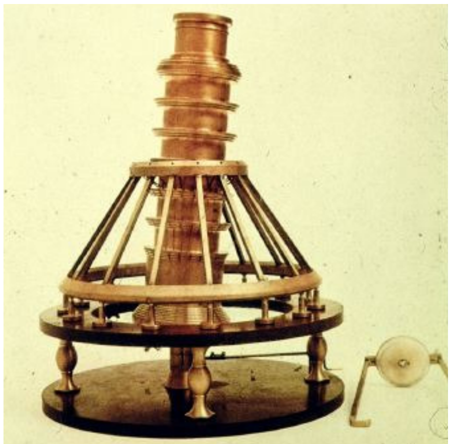


Figure 2-7 (a) York Cathedral; (b) Supporting rings for Pisa Tower (Mazzolani) [159]

Consolidation of Palazzo Carigliano in Turin (Figure 2-8). A provisional support system consisting of hot rolled profiles was designed to support the stone columns at the entrance hall during the consolidation operations of replacing the old floors with new ones. Steel profiles and corrugated steel sheets acted as formwork for the concrete cast slab [133]

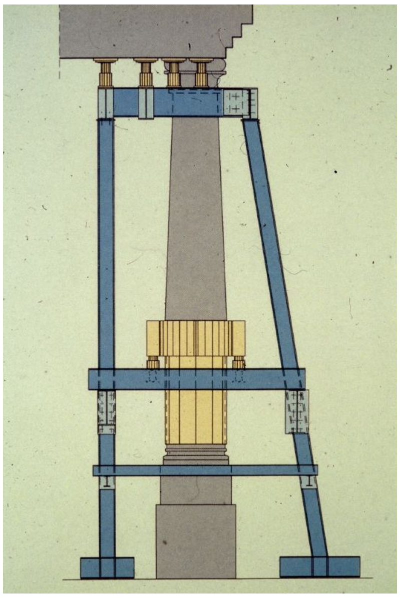
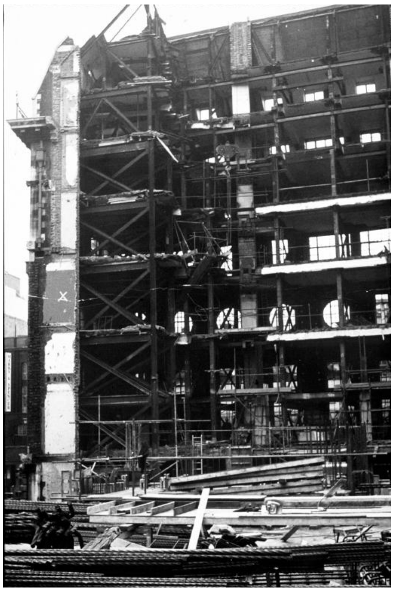


Figure 2-8 Palazzo Carigliano [133]



(a)

(b)

(c)

Figure 2-9 Waring and Gillow store (Mazzolani) [159]

Rebuilding of the Waring and Gillow store in London (Figure 2-9). Steel columns and girders acted as a bracing system used to preserve the old façade. Once assembled, these elements created an „ad-hoc” structure that supported the façade and a 5 m area behind it during the demolition of the remaining parts of the building. After interior reconstruction, in conjunction with current functional lay-out, the steel elements were gradually removed.

Restructuring of the old Muller theatre in Darmstadt (Figure 2-10). Horizontal steel rings fixed to the perimeter walls were used as temporary bracing that kept the walls of this building together while it was transformed into State-record Office.

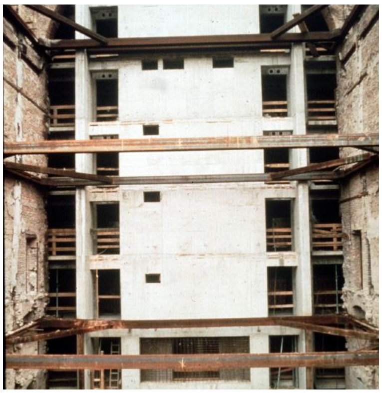


Figure 2-10 Darmstadt theatre [159] (Mazzolani)

Figure 2-11 Lisbon building (Mazzolani) [159] by courtesy of F.M. Mazzolani

Restructuring a building in the centre of Lisbon (Figure 2-11). A major fire led to the empting of a large masonry building in the centre of Lisbon. The remaining façades and the internal core were temporary supported by means of steelwork and perimeter ties prior to the restoration and consolidation operations.

Max Plank Institute in Rome (Figure 2-12). A complex consolidation operation for an old building in Rome required the temporary support of the façades by means of steel structures specially designed for this purpose before the degutting operations began [130].

(a)

(b)

Figure 2-12 Max Plank Institute (Mazzolani) [159]

Temporary support of the underground station walls in Naples (Figure 2-13). Tubular steel struts were erected for supporting the perimeter walls in order to protect the archaeological findings of the old roman harbour, during the execution of a new underground station.

Figure 2-13 Naples underground station (Mazzolani) [159] by courtesy of F.M. Mazzolani

2.2.1.2. Repairing

Repairing follows in chronological order after provisional safeguard and involves a series of operations to restore the buildings to its former structural efficiency. In essence it is about restoring the “status quo ante” of the retrofitting elements [133].

The repair takes place when the safety of the building is compromised by damages sustained due to earthquakes, atmospheric agents, or simple aging. As opposed to safeguard, repairing is a definitive operation and is normally used when the degradation of the structure can be easily diagnosed and comes from long term use or old age and doesn’t require urgent intervention. This is why there are cases when the economic aspect comes before safety aspects, when simple restoration of the structural performance is carried out without any strengthening of the construction. There is the possibility of making a simple repair even in the case of severe earthquakes when the lack of funds doesn’t allow a large scale consolidation.

From the point of view of safety, repairing offers a simple restoration of structural performance without introducing supplemental elements or strengthening of the building.

Fields of use and examples of application

Interventions on buildings affected by atmospheric agents and the passing of time (Figure 2-14, Figure 2-15, Figure 2-16, Figure 2-17).

In repairing there are a series of consolidation techniques based on the use of steelwork to improve the behaviour of all types of structures from steel to masonry and reinforced concrete or timber [133]. Beside steel, there can also be used for repairing, other traditional materials like masonry, reinforced concrete and timber.

The most commonly used materials in repairing operations are steel and more recently fiber polymers, which are considered modern and high performance materials.

Structural steel elements offer, by means of „prefabricated" technology, „ad-hoc" solutions designed to achieve optimal results and tailored to meet specific requirements. [139].

Figure 2-14 Masonry building damaged in time (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-15. Old wooden floor (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-16. Damaged wooden structure(Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-17 Corrosion phenomena in steel structure(Mazzolani) [159] by courtesy of F.M. Mazzolani

The specific requirements mentioned above can be detailed in the case of steel as follows:

- reduced weight that allows for easy transport, easy handling and erection which are important in conditions that require manoeuvring in the narrow spaces of historical centres of old towns;
- reversibility, thanks to the use of bolted joints, that allow the reuse of the structure after dismantling;
- speedy erection, a very important characteristic in the case of emergency repairs when damage spreads quickly;
- economically convenient, thanks to the possibility of re-use;
- modern feature clearly identified with the additional advantage of the use of technologies and materials that can be removed at any time without damaging the building.

Recently, the use of *fibre reinforced polymers* (FRP) has greatly increased in the repair of existing buildings. This material is one of the new materials, of great innovation, that are studied in recent times.

FRP are used as reinforcement bars for concrete or masonry structures or as strips for the strengthening of certain structural elements.

The most important characteristics of FRP are:

- excellent strength/self-weight factor, approximately 40-50 times higher than steel;
- flexibility, easy to shape and use;
- corrosion-free;
- highly resistant to fatigue;
- reduced weight – easy transportation and use and does not bring additional load to the building;
- short time of application;
- durability – no special maintenance is required and can withstand aggressive environments;
- satisfactory fire resistance;
- aesthetics – can be of different colours and textures.

The development of FRP is mainly justified by the absence of the corrosion phenomena that affect both steel and reinforcement in the case of reinforced concrete structures. The disadvantages of this material are its fairly high price, lack of ductility and insufficient research of its behaviour in time. In the matter of ductility those who promote FRP support the existence of a global ductility, in the absence of the material ductility.

Masonry structures

Vertical structures made of masonry, like columns or walls can be repaired using one of the methods that will be described below [146]. Repair methods aim to improving the behaviour of the element by restoring its load bearing capacity to gravitational loads but also to horizontal loads that come from the failure of foundations, geometrical asymmetry and seismic action. These methods in this stage of the consolidation do not aim to improve the dynamic behaviour but only to achieve local repair of certain elements. In this paragraph only masonry structural elements will be discussed, the partitioning elements being neglected. The retrofitting of masonry walls will be discussed in detail in Chapter 2.3.2. Only general considerations are presented in this chapter.

Improving the bearing capacity to vertical loads

<u>*With the help of steel elements:*</u>

- Confinement of the damaged masonry column by vertical angles and horizontal plates or by a combination between them in the case of columns with rectangular cross section, and with rings in the case of circular cross sections of the columns (Figure 2-18).The purpose of this method is to achieve a tri-axial state of stress by preventing lateral deformation, which results in a significant increase in resistance to compression and in the ultimate ductility of the consolidated element. The steel system prevents the instability phenomena of the masonry column and reduces cracks that occur parallel to the force direction. Steel elements must be fitted with a precompression force that increases the effectiveness of the method. An old method of achieving this precompression was to apply the steel rings at a high temperature. Nowadays the use of circular rings tightened with bolts leads to the same result.

- Inserting a new steel element along or inside the consolidated element, that can be either a column or a wall (Figure 2-19). The new inserted elements take up completely or partially the loads that belong to the vertical element. All types of steel, hot or cold rolled steel products can be applied successfully for this method.

- Inserting a steel frame, inside or around the opening of a wall, to restore the initial strength of the wall (Figure 2-20).This frame takes up vertical loads (acting as a lintel) and gives the masonry panel higher resistance to horizontal loads, achieving a unitary behaviour.

Figure 2-18 Consolidation of a masonry column (Mazzolani) [159] by courtesy of F.M. Mazzolani

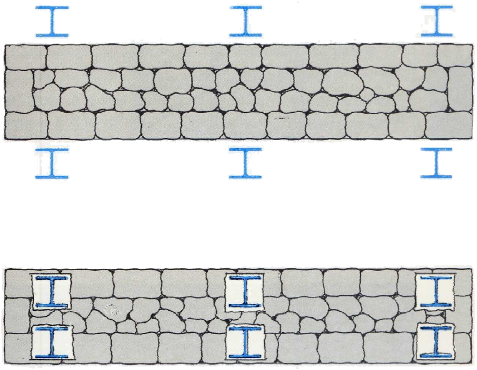


Figure 2-19 Consolidation of a masonry wall (Mazzolani) [159]

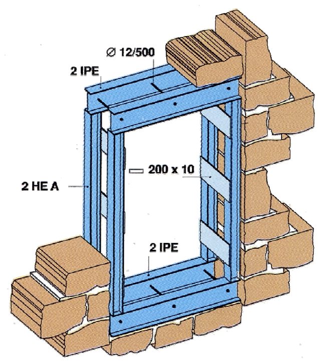


Figure 2-20 Steel frame used for a wall opening (Mazzolani) [159]

Figure 2-21 Repair of a reinforced concrete column by means of steel angles and ties (Mazzolani) [159]

Using fiber reinforced polymers:

Repairs can be done by means of polymeric materials used as reinforcement bars or as strips. The most used fiber reinforced polymers are **CFRP** – *Carbon Fiber Reinforced Polymers* – which comes in strips, glued with epoxy resins on the surface of the consolidated element. For the use as reinforcement bars they are called **NSMR** – *Near Surface Mounted Reinforced*. NSRM are placed in channels that are cut into the consolidated element and are covered with concrete.

Improving load bearing for horizontal loads

- Securing the façade walls with horizontal steel beams connected to transversal walls by means of ties that in the same time confine each floor in an attempt to tie the walls together so to achieve rigid body behaviour of the floor.
- Securing the corners of the building by means of steel columns connected by a system of beams and ties achieving the unification of the walls and making the transfer of horizontal loads to the walls that are parallel to the direction of the possible earthquake. Thus a confinement of the building is obtained and the rigid box behaviour of the building is achieved.

Reinforced concrete buildings

In recent times, most of civil buildings, both apartment buildings and office buildings, state institutions, schools, hospitals, have been constructed out of reinforced concrete. A classification function of the resistance structure divides reinforced concrete buildings into two categories: frame structures (with beams and columns as main resisting elements) or structures with reinforced concrete bearing walls – diaphragms. In the recent past a great accent have been put on the development of prefabricated reinforced concrete structures, and, nowadays, a great number of buildings exist that have been built by means of this technology.

Frame structures

<u>*With the help of steel elements*</u>

The methods used in the case of reinforced concrete elements are similar to those used for masonry and are based on the same principles, that is by achieving a tri-axial state of stress, meaning confinement of the element, and preventing it from falling apart. There are multiple measures to repair *columns* that have the effect of restoring the initial resistance to horizontal action but also to vertical actions. Some of these measures are mentioned below [136]:

Reinforced concrete columns can be consolidated with steel angles connected by steel profiles or steel ties (Figure 2-21) or by welded plates, thus achieving a higher degree of confinement (Figure 2-22). In the case of circular columns steel rings bolted together are used. In this case steel elements can be concealed by covering with a concrete layer.

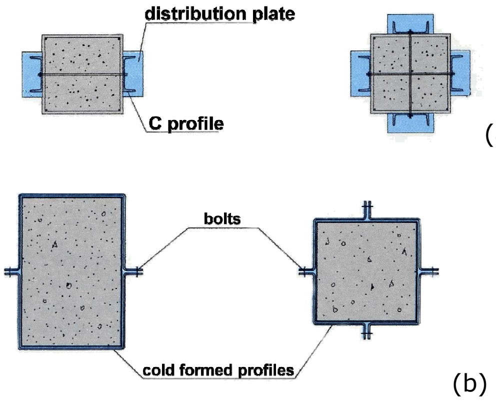
When the intervention must be visible, steel elements can be bolted to the existing column, and the steel ties that are used in connecting the steel profiles (U) go throught the reinforced concrete (Figure 2-23a);

Cold formed steel profiles with bolted connection placed around the column give the same result (Figure 2-23b).

The *beams* can be repaired in one of the following ways presented below:

The plating of beams with steel angles and batten plates is used in order to improve the resistance to bending and shearing of the beam and also of the connection between columns and r.c. beams. Welding, bolting or gluing by epoxy resins are used to ensure the connection between the steel elements and the consolidated r.c. element. The method is similar to the one used for columns or elements subjected to compression, but an increase in flexural behaviour is also desired. The flexural resistance modulus is significantly increased by enlarging the section and attachment of longitudinal steel elements to the extreme fibers of the beam. Moreover horizontal steel plates have a significant role in improving shear performance. Additional elements can be compared, from the point of view of their function, with longitudinal reinforcement of the horizontal elements along the beam and the vertical transversal elements with stirrups.

Figure 2-22 Steel angles used at consolidation of r.c. columns (Mazzolani) [159] by courtesy of F.M. Mazzolani



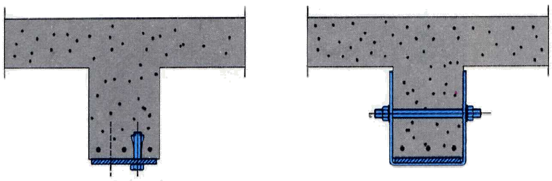
(a)

(b)

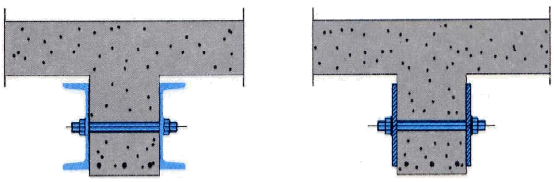
Figure 2-23 Increasing r.c. column section with hot rolled profiles (a) and thin walled profiles (b) (Mazzolani) [159]

Vertical ties that go across the upper part of a reinforced concrete slab and bolted to a bottom steel plate act like additional stirrups for the existing r.c. beam. The way this system works is similar to the system described before.

Cold formed steel profiles or plates can be placed at the lower part of the r.c. beam. Steel profiles are fixed by means of bolts or epoxy resin or by adhesive mortar (Figure 2-24). The gluing process has the advantage that unlike bolted connections it doesn't introduce stress concentration in the concrete element.



(a)



(b)

Figure 2-24 Steel plates or profiles fixed at the bottom (a) or lateraly (b) on the beam by means of bolts or epoxy resins (Mazzolani) [159]

With the help of fiber reinforced polymer

One of the modern means of repair consists in the use of FRP in order to strengthen the r.c. beams (Figure 2-25) and columns (Figure 2-26). Prior to the application of the FRP the support layer needs to be prepared, with sand or water under pressure, by degreasing and drying of the surface, and the use of special adhesives with epoxy resins. This method is easy to use and has the advantage of quickness of application. The application of the FRP can also be done by gluing the strip on a pretension element or achieving the pretension with the help of the strip. In this case the end zones of the strips require special attention.

Figure 2-25 Repairing intervention on a r.c. beam using CFRP (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-26 Repairing intervention on a r.c. column using CFRP (Mazzolani) [159]

In the case of columns the aim is to obtain a tri-axial state of stress.

Prefabricated FRP angles that are different in shape and obtaining process than the normal strips are often used for beams. The use of these elements requires sufficient anchorage of the FRP in the compressed zone of the element.

Steel structures

Steel structures can be divided, from the consolidation point of view, into new and old steel structures. The difference between these two types consists in the different construction method and profiles used. Two different means of approach can be distinguished.

In the case of structures built with materials and technologies that are still used nowadays, the consolidation technology focuses on integrating new steel elements. Methods with a high degree of generality and applicability can be established due to the modular characteristics of steel and to the existence of standard profiles.

One of the following methods can be used for repairing new steel structures:

The increase of the member cross section by welding or bolting new structural steel elements results in an improvement of the strength and rigidity characteristics (Figure 2-27). New elements can either reinforce or increase the area of the flanges or of the web in function of whether an increase of resistance to bending or to shear is desired.

Stiffening the structural nodes by introducing steel profiles, plates or stiffeners.

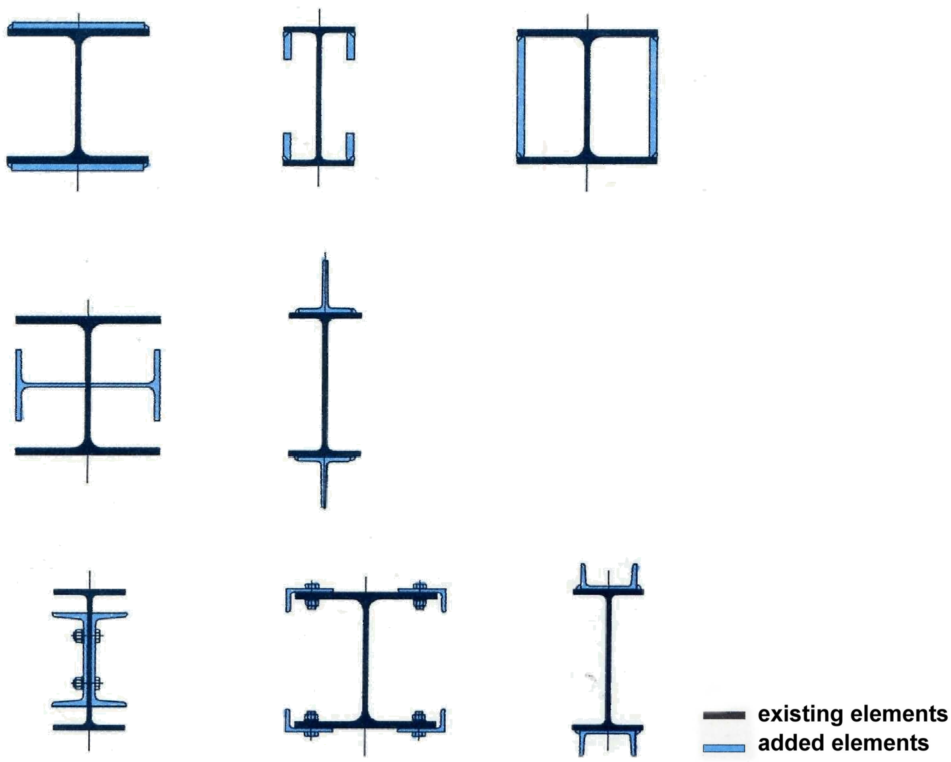


Figure 2-27 Repairing of steel cross sections by welding or bolting new elements (Mazzolani) [159]

Floor structures

Floors have mainly vertical load bearing capacity but the rigid diaphragm behaviour is a prerequisite in the case of any consolidation work. So as a consolidation must provide, from the antiseismic point of view the diaphragm behaviour. The floor types that are usually subjected to consolidation are the following [135]:

- Floors with wooden beams;
- Floors with steel beams;
- Floors with reinforced beams and tiles.

Wooden floors can suffer degradation and deterioration with the passing of time: rotting caused by atmospheric agents, loosening of the supports due to vibrations, weakening of the link between the floor and other structural elements etc. As an alternative to completely replacing the *wooden floors* one of the two methods described below can e used:

Working from the *bottom upwards*, repairing each wooden beam in order to increase their strength and reduce their deformations. Following this procedure, the following methods are used:

Steel beams are placed in the middle of each wooden beam (Figure 2-28);

Placing a pair of steel profiles (double T, U, angles, cold-formed) on each side of the wooden beam (Figure 2-28); in some cases they must be integrated by means of steel sheets or ferro-cement coating in order to support secondary elements (Figure 2-29).

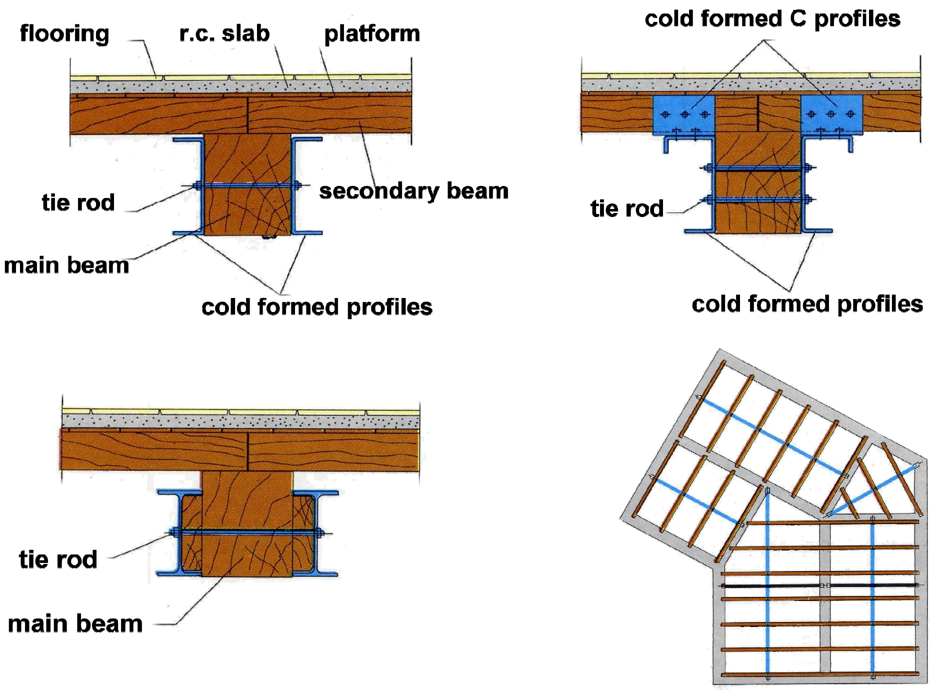


Figure 2-28 Repairing a wooden floor from the bottom upwards (Mazzolani) [159]

Figure 2-29 Example of intervention on a wooden floor (Mazzolani) [159]

The solution of working from *the top downwards* is chosen when the wooden beams are in a good condition and they may remain visible. This solution can be used in the following ways:

A steel beam is attached at the upper part of the wooden beam, linked by an appropriate connection system (connectors) to ensure that the two elements work together (Figure 2-30). If the connectors are correctly dimensioned so as to prevent slipping, a mixed steel-wood system is achieved that has improved strength and rigidity.

A more complex alternative to this solution is by creating a multiple composite system: the reinforced concrete slab together with the profiled steel sheet work together with the steel beam by means of welded stud shear connectors. In the same time it is assured that the steel beam works together with the wooden beam. Thus a composite steel-concrete- wood system is achieved (Figure 2-31).

In both cases presented, the achieving of a satisfactory collaboration between the steel and the existing wooden element is difficult because a connection with enough strength and rigidity to shear is difficult to achieve. That is why the finding of an appropriate solution for the connection is very important.

Modern methods propose a system that uses glued steel elements (bars or plates) completely hidden in the wood. Although these new systems are highly expensive, they present a significant advantage: good rigidity of the connection, ductile design of the joints due to the yield of steel, protection of the connection against aggressive factors, better architectural look.

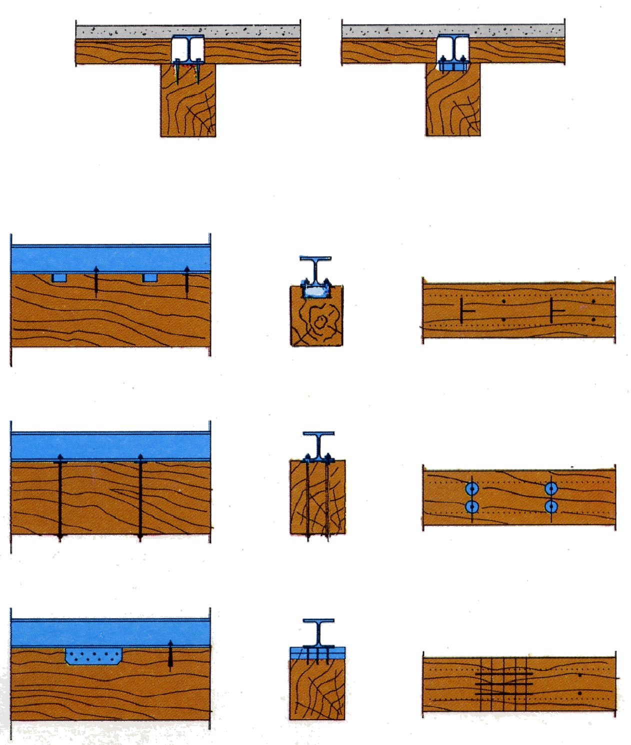


Figure 2-30 Repairing wooden floors by steel beams and connectors that create a composite steel-wood system (Mazzolani) [159]

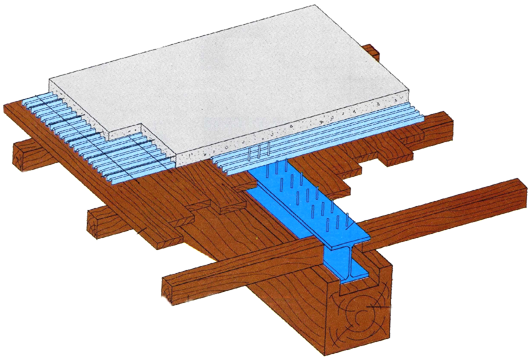


Figure 2-31 Repairing a wooden floor by using a multi-composite system (Mazzolani) [159]

Wooden floors are also characterized by weak horizontal rigid diaphragm behaviour, a fundamental requirement in modern antiseismic design. This behaviour can be improved by installing steel braces or profiled steel sheeting, or by casting a concrete slab that significantly increases the rigidity of the floor. However this procedure requires that the floor should be in a good condition and correctly dimensioned.

The *floors with steel beams* frequently used at the beginning of the 20th century, are composed of I shaped steel beams together with brick or tile vaults with concrete. Due to the degradation of the restraint condition, an increase in rigidity is often required. This can be done by using one of the techniques bellow:

- Working from the *bottom upwards*, the section modulus can be improved by welding to their bottom flange steel sections like plates, box sections, upside down T, double T etc. (Figure 2-32). In addition, cold formed profiles and omega profiles, can be attached by means of bolted connections.

- Working from the *top downwards*, a reinforced concrete slab can be connected by means of appropriate connectors (stub, angles, T etc.) so that it should collaborate with the existing floor slab. Additional steel elements can also be placed at the top part of the steel beam (Figure 2-32) or the section modulus can be increased by means of CFRP strips.

Composite slabs or tile floors can be repaired using one of the methods bellow:

- Repairing the r.c. beams by adding new reinforcement and special concrete after the degraded parts have been removed;
- Gluing steel plates or FRP at the bottom part of the r.c. beams (Figure 2-33);
- Inserting steel I profiles between the r.c. beams.

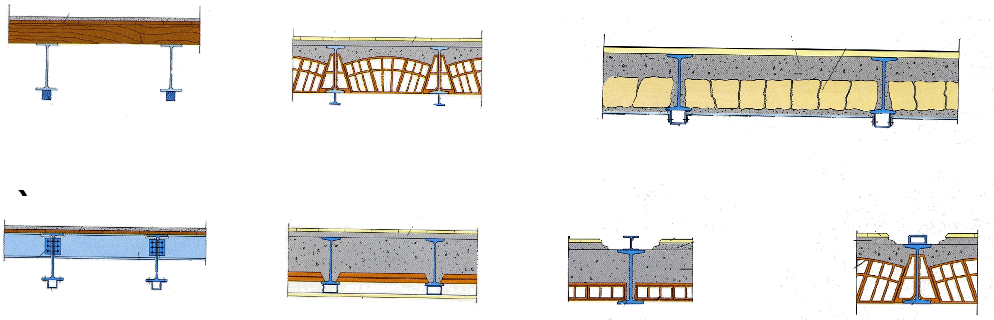


Figure 2-32 Increasing the steel beam resistance by welding additional elements (Mazzolani) [159]

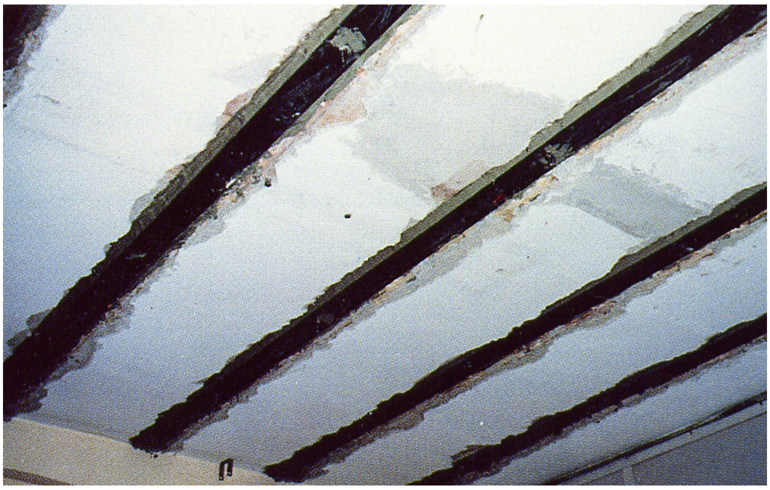


Figure 2-33 Repairing by attaching steel plates (Mazzolani) [159]

Roofing structures

Roofing of masonry structures is generally made of timber trusses which are often deteriorated due to the permanent contact with atmospheric factors. If the wooden elements are in an acceptable condition, a repairing solution can be found:

The repair operation consists in covering the wooden elements with steel plates or cold formed profiles with bolted connection or glued with resins. Such an example of repair, by using cold formed steel profiles and steel stiffeners at joints, bolted or riveted to the old structure, can be seen in Figure 2-81.

The use of CFRP, in the form of strips or reinforcement, continues spreading, although it has no solid economical justification.

If the wooden structure is in an advanced state of deterioration, the optimum solution is to replace the whole structure with new steel trusses integrated by trapezoidal sheeting and concrete casting. This system was used in many cases for the replacement of roofing structures of churches damaged by earthquakes (Figure 2-35 a and b).

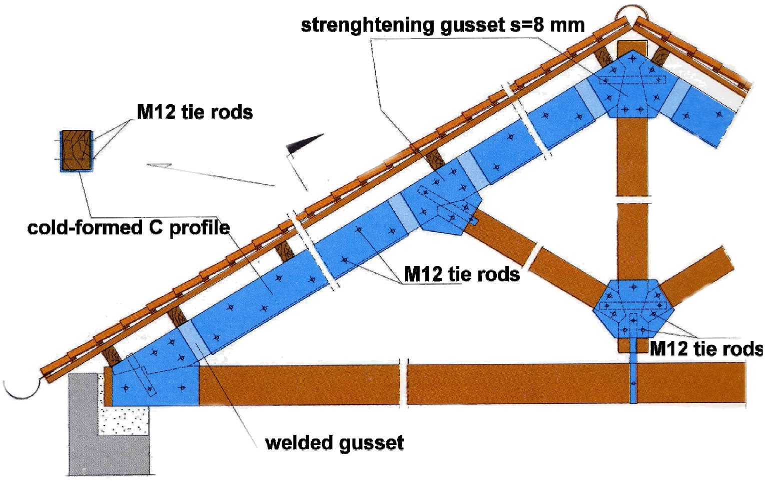


Figure 2-34 Repairing a wooden roof structure by using steel plates (Mazzolani) [159]

Figure 2-35 New steel roof structure of a restored church (Mazzolani) [159] by courtesy of F.M. Mazzolani

2.2.1.3. Reinforcing

It is the next step after repair, and it does not necessary imply a state of emergency of the building, but rather has the purpose of updating the bearing capacity and giving the structure the capacity to face new functional demands (new loads resulted from the change of functionality) or the change of position (as inclusion in a more severe earthquake risk zone) [133].

The consolidation does not produce any significant change in the structural scheme but brings new structural elements that need to be integrated with existing elements wihout changing or significantly altering the mass and rigidity distribution of the structure.

Unlike a simple repairing intervention, the consolidation can be made at different performance levels that depend on the level of safety that is desired to be achieved. This makes the consolidation operations fall into two categories from a seismic point of view:

- simple improving;
- seismic upgrading and adaptation.

Consolidation for improvement includes interventions on the whole building or just a part of it, with the aim of achieving a higher degree of safety without excessively modifying the static scheme and the global behaviour of the building. Improvement interventions repair elements in order to prevent local deficiencies that appear because of inadequate structure designs or execution (Figure 2-36 and Figure 2-37).

Consolidation for adaptation consists in a whole complex of operations necessary in order to assure that the structure has the required seismic capacity as presented in antiseismic codes. This can also include the change of the static scheme with complete modification of the global behaviour (Figure 2-38).

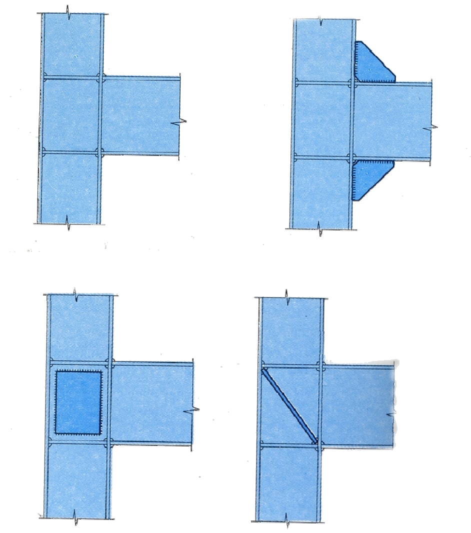


Figure 2-36 Improving operations on a beam to column connection by adding welded parts (Mazzolani) [159]

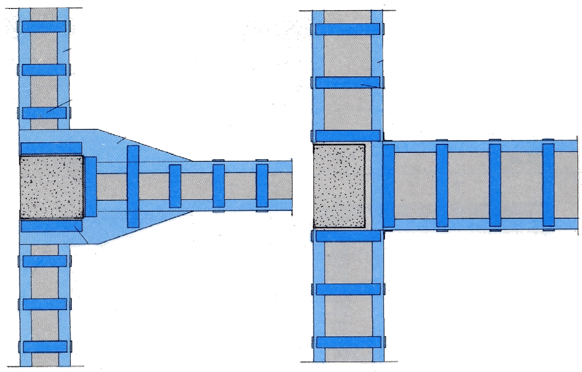


Figure 2-37 Improvement operations on a r.c. node by means of steel angles, plates and battens (Mazzolani) [159]

Fields of use

Improvement is applied in the following situations:

- Buildings under the condition of load increase due to the change of destination of the building which leads to an increase in live load;
- Existing buildings that were included in a new seismic zone or for which the conditions of determining the seismic loads were changed, leading to an increase of seismic load.

For monumental buildings seismic conformation, especially in the case of big churches, there raise a big number of problems due to large free spaces and painting on the walls that could suffer and loose their artistic value if extended consolidation works are executed.

For all these, the design codes foresee distinct conditions for structural improvement or for a rigorous anti-seismic conformation.

This last provision can be applied in the following cases [139]:

- increase of loads and functional demand changes as a result of the change of destination;
- for storey adding or extending, which results in the increase of volume and area;
- when the necessary changes for consolidation essentially modifies the static scheme of the existing building or the global behaviour.

The conformance can be avoided only if the loads are not modified or if the consolidation operations are made for monumental buildings. In these cases it is enough to limit to the amelioration interventions, which can include techniques with a reduced impact on the existing building.

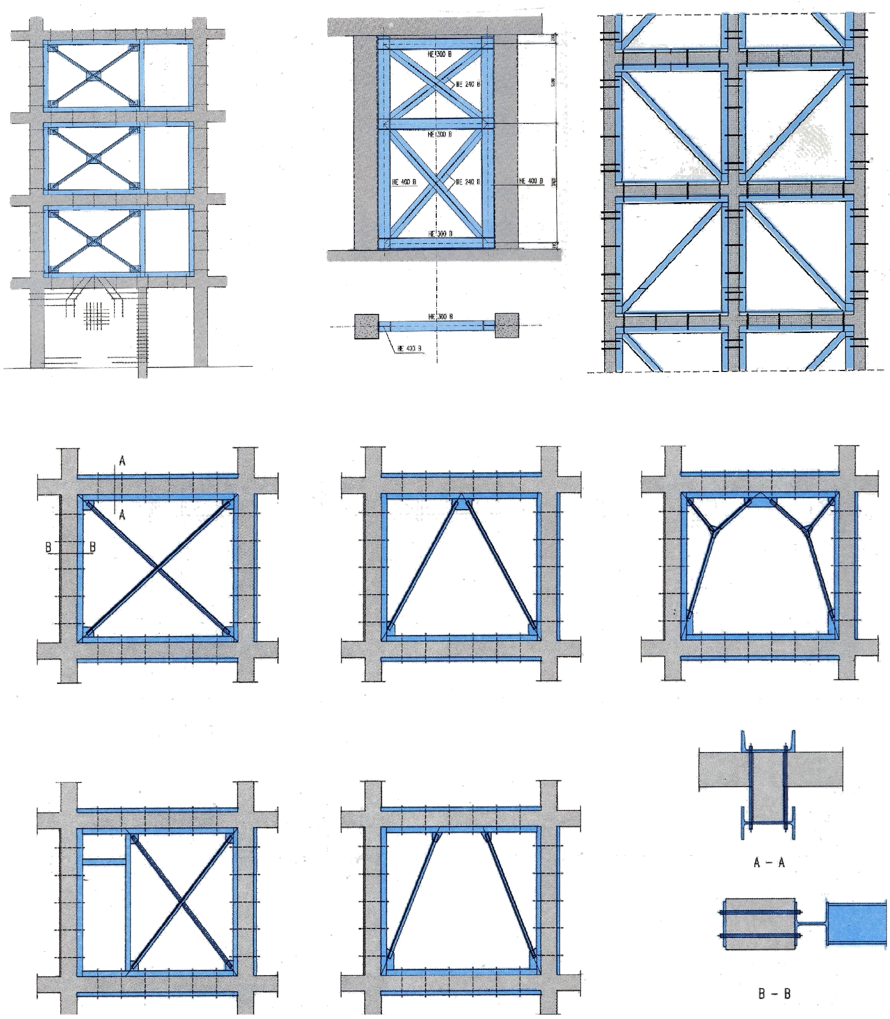


Figure 2-38 Metallic braces systems for seismic conformance of reinforce concrete frame (Mazzolani) [159]

Technical details of intervention

In order to obtain different levels of strengthening, from the simple improvement of elements behaviour to the modification of global response, the same techniques can be used as for repairs, but with a higher degree of consistence and a much larger generalisation.

The solutions based on steel are used widely to obtain an improvement in static behaviour of brick masonry or reinforced concrete structures.

Lately, improvement solutions based on FRP have been used with good results for brick masonry or reinforced concrete structures.

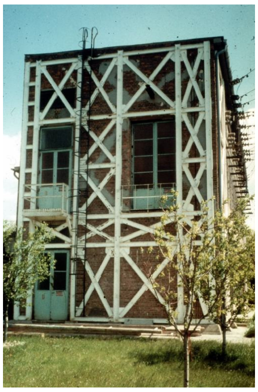


Figure 2-39 Electrical power plant, Hungary (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-40 Church in Avelino (Italy) (Mazzolani) [159]

Figure 2-41 Block of flats in Santa Monica (California, SUA) (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-42 Building in Tessaloniki (Greece) (Mazzolani) [159] by courtesy of F.M. Mazzolani

Metallic braces systems are widely used for anti seismic conformance for the structures mentioned before, (Figure 2-41 and Figure 2-42) [136]. These systems are supposed to optimise the structural response by increase of strength and rigidity of the structural assembly for horizontal forces. Systems based on centric or eccentric braces can be used. The use of centric braces (St. Andrew's cross Figure 2-39) offers a significant increase of rigidity and a negligible weight increase. For some cases, less rigid solutions are suitable, like eccentric braces which offer considerable increase in ductility. One of the advantages of the system is the fact that no new independent foundations are required, but a re-evaluation of the existing ones for the new global behaviour of the structure. Special attention must be given to the connection of the concrete element with the metallic system because the connection is the vulnerable element during an earthquake.

In the category of systems based on braces also fall new systems that use modern technologies: steel buckling restrained bracing – BRB (Figure 2-44), braces that are made of shape memory alloy bracing – SMA-B (Figure 2-45), or low yield steel or pure aluminum stiffened panels (Figure 2-46) (Experiments done in Bagnoli, Naples, Italy – ILVA Project) [149].

Figure 2-43 Reinforced concrete structure consolidated with EBF (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-44 Reinforced concrete structure consolidated with BRB (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-45 Reinforced concrete structure strengthened with SMA braces (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-46 Aluminum stiffened panels (Mazzolani) [159] by courtesy of F.M. Mazzolani

The accepted technologies for consolidations can be schematically represented in the matrix desribed bellow. It can be observed that not all the materials are suitable to be used for the consolidation of structural types defined by the material used. The use of materials different from the base material can create the so-called mixed technologies based on the composite action of different materials, which form new mixed materials. (Table 2-2):

Table 2-2 The structure – material relation [159]

New composite materials						
Structure to be consolidated		Materials used for consolidation				
		Metal	Concrete	Masonry	Wood	FRP
Type	Metal	XX				X
	Concrete	XX	X			X
	Masonry	XX	X	X	X	X
	FRP	XX			X	X

There can be noticed, from this table (Table 2-2), the impossibility to combine all the materials, but the composite action of two different materials can result in a behaviour superior to individual materials.

Not all the possibilities presented before fulfill one of fundamental demands in modern consolidation philosophy, i.e. the reversibility. This requirement becomes very important in the case of monumental buildings.

If the designer decides to use irreversible techniques, it is very important to take into account the following aspects:

- *Compatibility* – mechanical properties for the material used for consolidation in relation with the structure (strength, deformability, thermal expansion coefficient etc.);
- *Durability* – refers to the new material characteristics that must be compared with the existing traditional materials.

Examples of application

Sigma Coating building from Agnano, Naples. The change of destination for an industrial hall required the change of capacity due to the increase of live load from 2 kN/m^2 to 20 kN/m^2. The existing structure was composed of reinforced concrete, two storeys, and frames with isolated foundations under the columns. The consolidation consists in [135]:

- the transformation of the foundations from isolated to continuous reinforced concrete foundations(Figure 2-47);
- the increase of the axial force capacity for columns by disposing supplementary cold formed profiles;
- the increase of the bending moment capacity for beams by disposing a 20cm concrete slab at the top and achieving the composite action, and binding at the bottom of a metallic plate, connected to the top through metallic ties (Figure 2-48).

Figure 2-47 Consolidation of an isolated reinforced concrete foundation (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-48 Consolidation of a reinforced concrete beam (Mazzolani) [159] by courtesy of F.M. Mazzolani

Reinforced concrete building at „University of California", Berkeley (USA).

Figure 2-49 The main university building Berkley (California) (Mazzolani) [159] by courtesy of F.M. Mazzolani

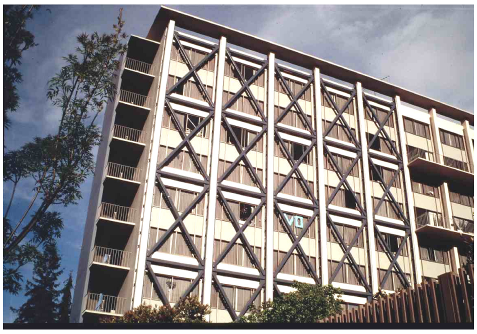


Figure 2-50 Apartments building for students, Berkeley (California) (Mazzolani) [159]

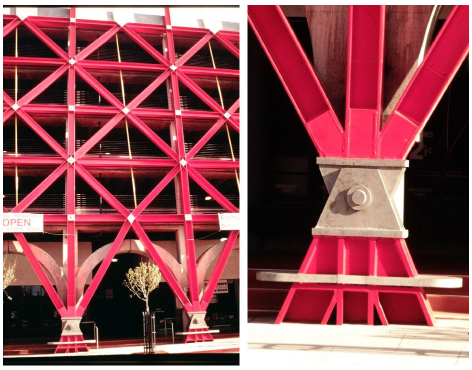


Figure 2-51 Car parking building in Berkeley (California) and detail of the base hinge (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-52 A steel mill in Bagnoli (Naples) (Mazzolani) [159] by courtesy of F.M. Mazzolani

After the „Loma Prieta" earthquake (1989) in California, many buildings from Berkeley were strengthened by disposing metallic braces systems [146]. The main university building is a significant example of metallic braces that form a network disposed over two storeys. This network increases the strength of the building and gives a new architectural aspect to the old façade. (Figure 2-49). The same intervention system, based on the same structural principles, was applied to the apartment building for students (Figure 2-50) and for a car parking building (Figure 2-51).

ILVA continuous casting mill in Bagnoli, Naples. The earthquake from November 1980 happened when the structure was in the erection phase (Figure 2-52), the structure being designed in accordance to old design codes. After this event, the location was included into a new seismic zone, so that the structure had to be modified in order to fulfil the new demands and it was compulsory to upgrade the structure. Following the calculus, it was established that the structure did have sufficient strength on longitudinal direction. Due to columns design, built in for transversal direction and pinned on the longitudinal one, it was possible to limit the intervention only to the longitudinal direction. The necessary structural intervention consists in connecting the columns bases through a metallic system and in increasing the capacity for braces. (Figure 2-53).

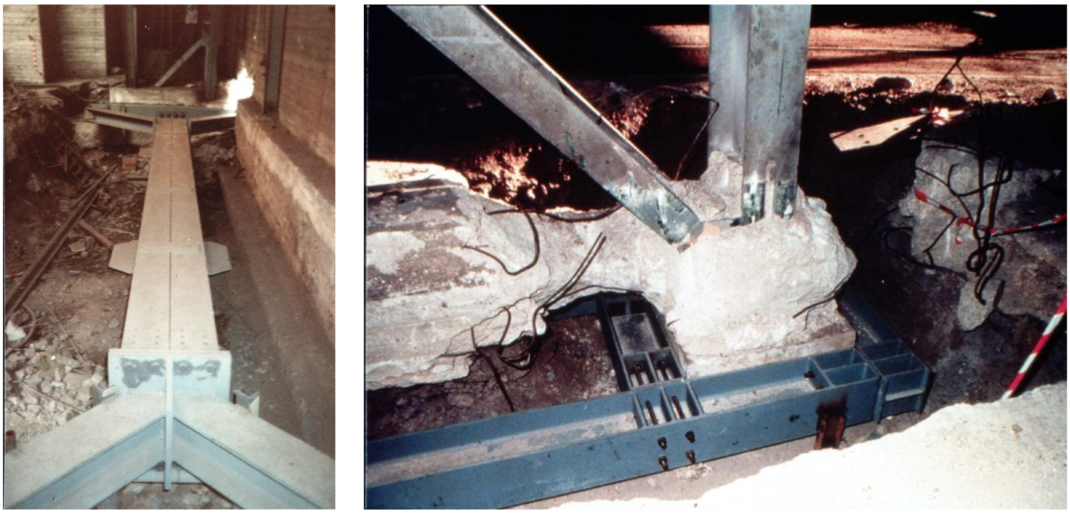


Figure 2-53 The connecting structures at the foundation level (Mazzolani) [159] by courtesy of F.M. Mazzolani

2.2.1.4. Restructuring

The most general level of consolidation is the restructuring, which is the partial or total modification of volume distribution and implies a radical change of the initial static scheme. This can be done when the destination change is needed, the creation of new functional spaces, new volumes, or when the imposed capacity level is impossible to be attained without a substantial resistance skeleton change (Figure 2-54 and Figure 2-55).

Restructuring interventions:

- interventions that substitute the internal part of a building with new structures of different types;

- vertical or horizontal additions of new volumes;
- introduction of new structures or structural elements into the existing structure;
- replacing of heavy elements with other lighter elements.

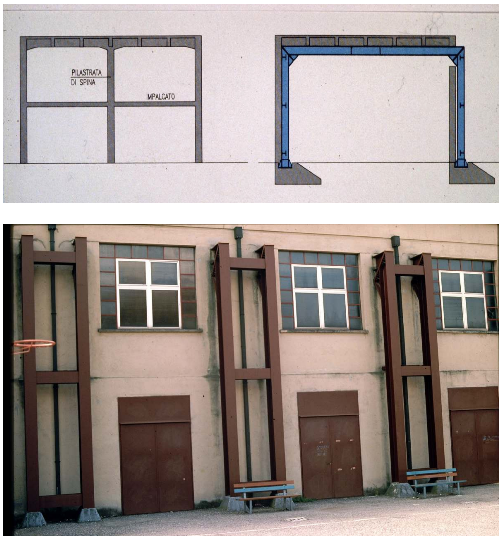


Figure 2-54 A new steel structure inserted in the original lay-out of a r.c. building, (Cantù, Como, Italy) (Mazzolani) [159]

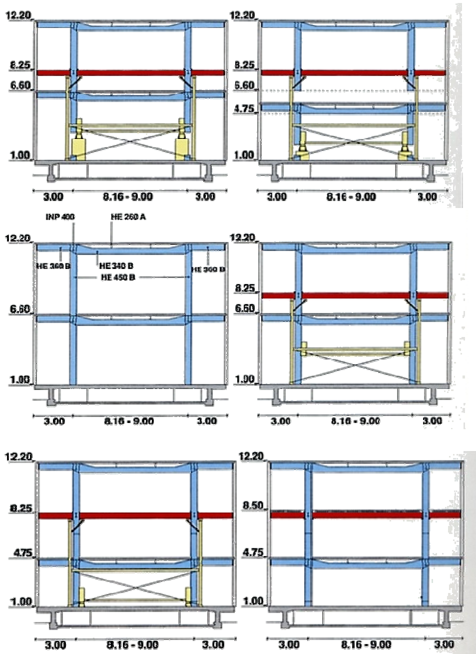


Figure 2-55 Insertion of an additional floor in an existing steel building (Amstelveen, The Netherlands) (Mazzolani) [159]

Historic context - past situation and actual tendencies

Cultural debates on forms and restructuring criteria of buildings are two centuries old and they started in the same time with the systematic study of materials and their use for constructions. At the origin of divergences are not only the structural aspects but also and mainly cultural and architectural issues. First of all, an ambiguity derives from the word "restructuring" itself – Lat. re-sisto which means – to build, to rise, to do, where the prefix re – can mean re-systematisation, recovery, "status quo ante", and even revision, transformation, reinterpretation intent to rise, in the past centuries conception, the necessity of adaptation to new static and functional demands.

When it comes to the preservation of monumental values, as a witnesses of past historic times, the procedure of restructuring, the creativity and the artistic and architectural gumption must be eliminated, in order to preserve the initial philosophy and background, excluding any modification possibility. The intervention must be centred upon restoring and optimising the resistance structure, as a simple static consolidation.

Nowadays, there is an attempt to refunctionalize constructions in such a way that the intervention should offer to the past a new vitality that increases the intrinsic value. The new problems are how to obtain and capitalize new spaces without hiding the existing elements. In this approach the opposite and contrasting forms and colours may be used, as well as different shapes, in order to put the old values in a new light. The rehabilitation work in this case can be seen as an architectonic matter.

After the Second World War, the general principles of rehabilitation have been stated in the following Charta from: Paris (1957); Venice (1964); Italy (1972); Conservation and restoration (1987).

These documents allows for restoration works an individual characterand the use of independent structures. This fact justifies all the restructuring interventions.

Restructuring interventions typologies

Depending on the intervention level required, the consolidation may be classified in two distinct categories:

- Conservation
- Remediation

Conservation intervention

This type of intervention is generally required when no modification of the destination is needed or the modification doesn’t change the internal spaces or the existing volumes. This applies when architectonical reasons don’t permit a substantial change of the original scheme, or the presence of art work as paintings, statues obliges to keep the initial shape untouched. This is why this intervention is named conservation and finds its application in the case of buildings with highly artistically or monumental values. The purpose of the intervention is to well preserve and offer a good resistance and functionality to the building that is in an advanced stage of damage.

Techniques that can be applied are related to elements consolidation, partial or total consolidation of structural components with or without upgrading the initial performances, but by keeping unchanged the static scheme and volumes of structures. Thus these techniques must harmonize with the existing structure without negatively affecting the architectonic balance. Steel can find a good justification in these cases, it being the optimal choice, mainly for technical reasons, not just economical. Since metallic materials shows high strength, it reduces the weight and dimensions, so thst the “dry” connection technique and easy the erection integrate well into the existing assembly. The intervention technique interferes as minimum as possible with the occupancy of the building.

Remediation intervention

It is needed when a new destination different from the initial one, is desired. It implies substantial modifications of the static scheme and the interior volumes. Due to this major modification of the static scheme, the load paths, and the interior partitioning, the load bearing structure can suffer important revisions. Sometimes this intervention may be confused with restoration intervention.

The main features of ameliorating interventions divide them into specific operations characterized by certain philosophies, methodologies and operations.

These intervention methods are classified into [133]:

- *Degutting;*
- *Insertion;*
- *Addition;*
- *Lightening.*

The first two imply the alteration of the initial structure, the load paths, the internal spaces, by building new structures, possibly statically independent, inside the existing building (*insertion*) or by total or partial substitution and replacement of an existing part of the structure by a lighter new one (*degutting*).

Addition modifies the external volumes by increasing them, when new spaces are needed for new functional demands.

Lightening, in opposition with addition, tries to reduce the volume by eliminating some elements, assemblies, and parts in order to create new spaces or to reduce some supplementary loads. Lightening replaces heavy r.c., masonry or wood elements by lightweight steel elements.

In opposition with the conservation intervention, in the cases of ameliorating interventions, the existing and the new elements coexist and emphasise the architectonic value of the existing building. The new elements mainly have the role of load bearing structure. Thus, the contrast between old building and new materials and technologies used for consolidation is obvious. Once again, steel perfectly meets these demands.

2.2.1.5. Degutting

It consists in the total or partial substitution of the internal structure of a building for the purpose of changing its destination and structural concept.

This is used when, for architectural and urban reasons, the preservation of the façades is needed or a much too drastic change of the structure is needed in order to meet new structural and functional demands.

The objects of "degutting" usually are buildings situated in the historic town-centres, when the preservation of the façades is desired for image reasons (as in the case of banks). This is preferred when the consolidation of the existing structure would be much too difficult or too expensive. In this case, a new independent structure, with its own foundations, is the support of the remaining façades, now without any static role, but preserved for aesthetic reasons (Figure 2-56).

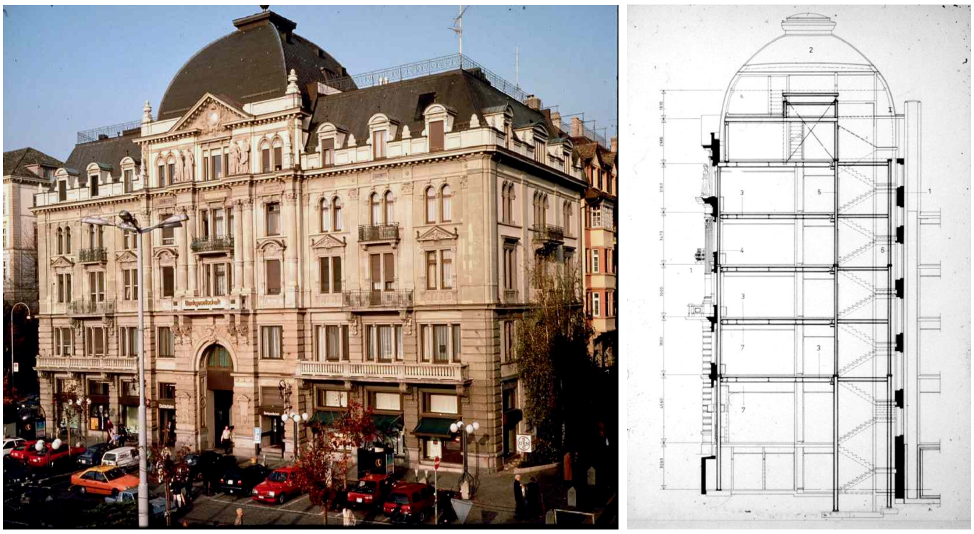


Figure 2-56 Example of degutting (Zurich, Switzerland) (Mazzolani) [159]

Modern philosophy imposes that the structure be made with modern performing techniques, new materials, which is again a good argument for the use of steel – resistant, performing adaptable to new technologies and flexible to any structural scheme that needs quick execution or high manoeuvrability.

Because of the reduced dimensions of the steel elements, the maximisation of space availability and thus of the functionality is obtained.

2.2.1.6. Insertion

All operations and interventions that imply the adding of structural elements inside the existing structure fall into this category.

A characteristic of this intervention is the maintaining of the added elements in plain sight, so that tey should have their own artistic expression. These operations must respect the architectural identity of the existing buildings.

Inseration is usually done when an improvement of the interior functionality is desired, thus giving birth to new usable spaces, or it can be conceived for the static optimisation of the existing structure, giving part of the structural load to the inserted structure (Figure 2-57).

Many structural and architectural solutions can be used for this type of intervention:

- introducing an intermediate floor with the purpose of improving and enlarging the free surface inside an interior volume;
- introducing an elevator shaft in order to improve the building feasibility and to satisfy new and more strict safety codes;
- structures for sustaining the roof or the upper floors, built with own foundations in order not to bring excessive loads to the existing structure;
- self bearing skeleton destined for museums or showroom equipment.

Figure 2-57 Example of insertion (Genoa, Italy) (Mazzolani) [159] by courtesy of F.M. Mazzolani

The use of steel is mandatory in the case of insertion interventions because of the need to preserve the character of the existing structure that requires the addition of slender, light, easily removable, and in some cases reversible structures.

That is why insertion of non-autonomous structures that unload on the existing structure requires the use of lightweight materials that do not overload the structure and have a reversible character, important when the provisional steel structure must be taken out.

2.2.1.7. Addition

It implies the building of another level on top of the existing ones, or near the existing building, destined to change the initial global volumes. This modification can be done on the vertical or on the horizontal direction.

This intervention is needed when new functional expectancies demand to create new spaces the function of which is in correlation with the destination of the existing structure.

When new levels are built on top of the building, the addition is called *vertical addition.* This is one of the most delicate cases of addition, it depends on the horizontal and vertical configuration of the new added masses and requires a careful examination of the static conditions of the existing structure in order to decide whether a consolidation is needed or not. From this point of view, the evaluation of the degree of safety can be very complex and difficult, especially in the case of structures built in the past century, the structural skeleton of which is comprised of masonry bearing walls and for which a highly accurate analysis must be made in order to find out all the morphological and resistance aspects of the existing structure.

Vertical addition

The addition of a level (Figure 2-58a) can be done if there is an surplus of resistance of the structure, or the choice of consolidation is made to give adequate capacity to the structure in order to withstand the new loads. This intervention is a very delicate one, given both the extending character of the building, which implies all structural elements, and because it is difficult to predict the actual behaviour of the structure with the new floors. The problem becomes even more pressing in seismic areas, where the evaluation of the global behaviour of the building under dynamic load is significantly changed by the presence of new inertial masses at the top of the building.

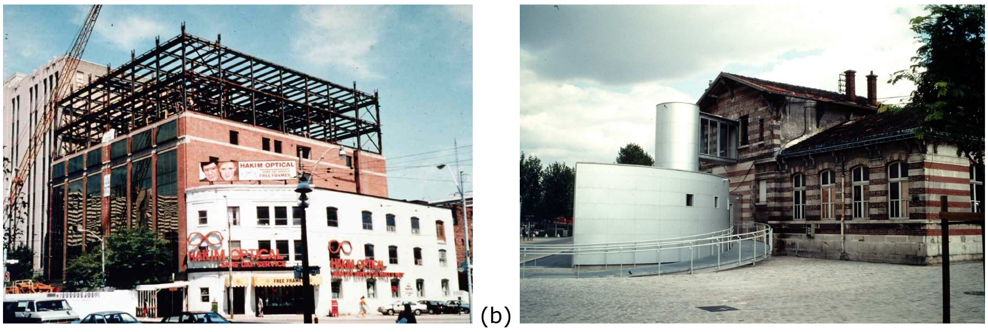
(a)  (b)

Figure 2-58 Example of Vertical addition, Canada) and horizontal addition (b. La Villette, Paris) (Mazzolani) [159] by courtesy of F.M. Mazzolani

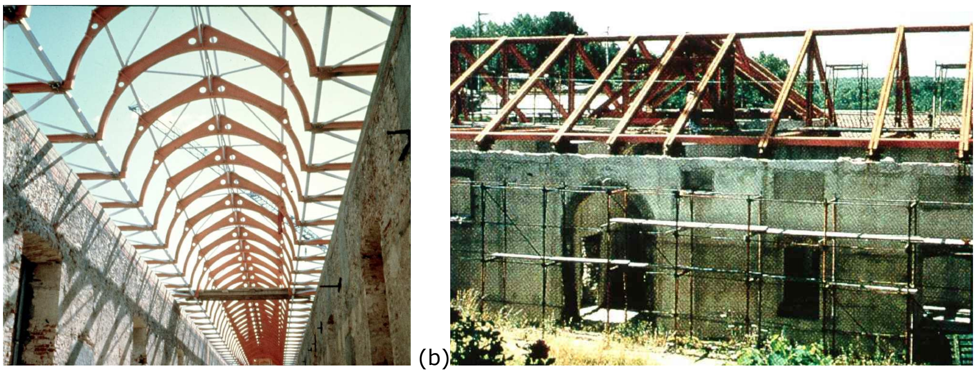
In this case it is advisable that the added structure should not interfere with the existing structure in order not to compromise its static equilibrium; the support structure for the vertical addition to be independent of the main structure and to transfer additional loads directly to the foundations by means of vertical bearing elements positioned inside or outside of the structure. This problem brings about the choice of the technological plan, in opposition with the choice of light materials with maximum structural and mechanic performance so as to modify as favourably as possible the behaviour of the structure. Once again, steel is an appropriate choice capable of offering a good and rational solution. When the existing structure is capable of taking extra loads, the reduced weight of steel materials, helps sparing the resistance reserve of the structure maximum. Even when the new floors are added on a static independent structure, the use of steel will prove correct by reducing the inertial masses concentration at the top.

Horizontal addition

Finally, when new volumes are added alongside the existing building (Figure 2-58b), in this case a horizontal addition, the intervention assumes in most cases the characteristics of the existing structure. In this case the problems arise more from the aesthetic point of view than the structural one, because of this need to attune very different architectural “languages”. Even in this case the use of steel was appropriate as many interventions of this type proved.

Weight reduction

For clear reasons, the weight reduction and storey adding are opposite, and it is possible to foresee total or partial demolitions of one or more storeys of the building. The goal is to reduce the tension state from the existing structural elements. Actually the term “discharge” can acquire a more ample meaning, generally any operation that somehow reduces the self weight of the building in order to improve its functional aspect. In this category fall the substitution operations, the roof replacement (Figure 2-59), and the replacement of coverings and even of all structural elements type.



(a) (b)

Figure 2-59 Exemples of weight reduction by roof replacement (a. The Rivoli Museum , Torino, Italia; b. Mongiana Factory, Italia) (Mazzolani) [159] by courtesy of F.M. Mazzolani

For example a partial emptying regarding locally the structural resisting case of the building by replacing the structural elements by lighter ones can fall into the weight reduction strategy.

This procedure has a great favourable impact on constructions located in seismic areas, by reducing the structural masses from the upper storeys and regulating masses in plane and by height, thus reducing in this way the torsion effects.

This type of interventions, which imply the replacement of structural elements and not their removal from the building, are practical when the old and heavy wooden roof carpentry is replaced with a new and light structure. Once again, steel proves to be suitable due to the low weight structures which can replace carpentry, roofs, stairs and envelopes, obtaining not just a lighter structure but a more efficient structure from the structural point of view.

It is particularly underlined the economical possibility to profit from the weight reduction which can be done by simply replacing the heavy or non efficient elements, replacing the carpentry with steel work, thus increasing the bearing capacity of the building: a consequent weight reduction can eliminate the necessity to consolidate other elements, which do not meet the requirements for the existing situation (vertical structure), thus saving time and money.

Domains of use and technical details

Restoration applies rationally in one of the following situations:

- when the modification of the structure destination and of the interior structure needs adding and the introduction of new spaces, volumes and areas;
- when the requirements imposed by the design code implies the modification of the resistance structure;
- in the case of highly structures for which the simple consolidation operations are not enough.

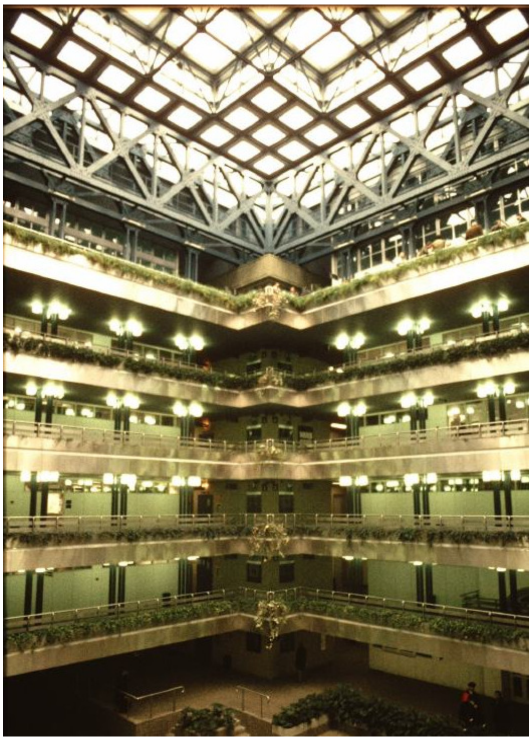
The choice of a method of consolidation must take into account the following aspects:

- if the object of the consolidation is a historical building, it is required to use reversible technologies;
- modern restoration theories and the preservation of existing buildings by integrating reversible works and with a clear individuality represent basic criteria for every intervention;
- steel and the applied technologies in steel works offer the answer to these demands by the modern character of reversibility and in particular by its ability to harmonize with old materials, thus building unitary structural systems.

Examples of application

Degutting (The Law Court Palace from Ancona, Italy)

The restoration of the Court Palace in Ancona is an emblematic example of emptying. The building was emptied and restored, and in the same time the masonry façades were maintained, preserving the new-Renassance style of the building. Four towers made of reinforced concrete of 9 by 9 m, including the stair case, the elevator shaft and the utilities, were placed at the corners of the area, and were limited by the remaining façades. The structural role of these towers was to overtake the vertical loads from the roof and from the suspended floors, as well as to offer the necessary resistance in the event of an earthquake (Figure 2-60). The system of four suspended floors is composed of steel beams and concrete slab reinforced with corrugated sheet. The floors are placed in four zones of 9x20m between the four towers. In Figure 2-61 it can be observed in detail the building roof made of a metal beams system [136].



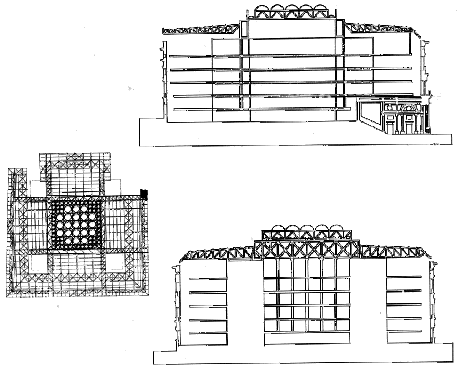


Figure 2-60 The tower and the suspended floors from the Court in Ancona, Italy (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-61 Trellis girders from the upper part of the Court in Ancona, Italy (Mazzolani) [159] by courtesy of F.M. Mazzolani

Insertion (Ducal Palace from Genoa, Italy)

During the past centuries the buildings have suffered many changes in destination, this fact leading to an advanced degradation of the resistance structures. Due to the fires of 1591 and 1977 and to bombardment from 1944 structural restoration was needed. The use of steel was justified by the need to differentiate the new and the old parts of the building so as to give a modern aspect to the building [136].

Particularly, insertion work was done as follows:

- new suspended ramp in order to create a connection between „Loggia degli Abati” and „Torre di Palazzo” (Figure 2-62);
- new amphitheatre made off curved steel beams.

Articulated system from service stair case and the Intermediary mezzanine (Figure 2-63a and b).

The suspended was ramp suspended from the upperr part of the palace roof, that was made of steel trellis girders.

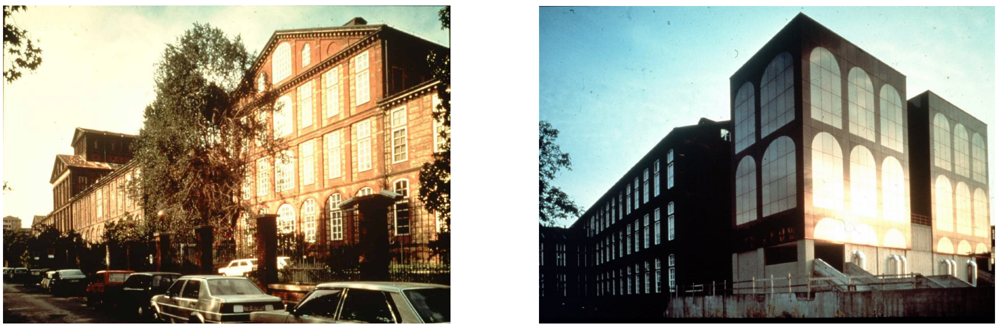
Each of the works is conceived in such a way as to interact with the structure as little as possible.

Figure 2-62 Suspended ramp from the Ducal Palace in Genoa, Italy (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-63 Amphitheatre (a) staircase and mezzanine (b) Ducal Palace in Genoa, Italy (Mazzolani) [159] by courtesy of F.M. Mazzolani

Horizontal extension (The economic Science and Marketing Faculty in Torino, Italy)

This work represents a lateral extension example of an existing building, a former retirement home for the elders (Figure 2-64 a). In order to create the mezzanine and the staircase inside the existing building, metallic confection was required, with respect to the reversibility demands of the consolidation works.



(a) [159] (b) [159]

Figure 2-64 The Economic Science and Marketing Faculty in Torino, Italy (a) before the intervention (b) after the intervention [146]

Moreover, new spaces were created and new volumes for the new amphitheatre. The new building is characterized by a modulated façade (Figure 2-64 b) made of panels which keep the same window configuration as the existing building. The principal problem that arises is the architectural harmonisation.

Vertical extension (Country Club in Briatico, Catanzaro, Italy)

The existing building is an ancient sugar factory (XIII sec), a relevant example of industrial archaeology. The restoration has the purpose to change the structure according to its new destination for social activities and expositions. The building has suffered important damage due to the last earthquake and soil erosion because of the sea (Figure 2-65 a). Due to the aforementioned conditions, the reconstruction solution with old methods was excluded. A new layer was created over the existing masonry walls, layer which transmits to the masonry underneath only the vertical loads, due to a special supporting system (Figure 2-65 b).

(a) (b)

Figure 2-65 Briatico building (a) before restoration (b) after restoration (Mazzolani) [159] by courtesy of F.M. Mazzolani

Vertical extension (Building in Toronto, Canada)

The original building had 6 storeys and was made of reinforced concrete frames (Figure 2-66 a). It was designed to be extended by four more storeys, also made of reinforced concrete.

(a) (b)

Figure 2-66 Building in Toronto, Canada (a) original building (b) building with added storeys (Mazzolani) [159]

After a few years the storey adding began, but steel was used instead of reinforced concrete. Due to the advantages offered by steel, instead of four storeys there were added eight storeys, the building becoming a construction with fourteen storeys (Figure 2-66 b).

Weight reduction (Cultural Center in Succivo, Caserta, Italy)

The former Carabinieri Barrack of Succivo was transformed into an antique shop and a Cultural Centre. The change of the building destination together with the static improvement demands, required the creation of additional new spaces with a contemporary lightening of the structural conditions. In order to fulfil both statical and functional requirements, the original roof structure has been replaced by a new metallic attic floor (Figure 2-67), composed of a series of Vierendeel trusses, with four vertical elements, tall enough as to accommodate a new suitable space (Figure 2-68). Despite a little increase of the overall building volume, due to a slight increase of the height of the roofing level, a sensible weight reduction has been obtained thanks to the use of light gauge steelworks.

Figure 2-67 New roofing of Cultural Centre Succivo, Italia (Mazzolani) [159] by courtesy of F.M. Mazzolani

Figure 2-68 New spaces created in penthouse (Mazzolani) [159] by courtesy of F.M. Mazzolani

2.2.2. New strategies in antiseismic protection

2.2.2.1. General criterias

The passive protection of constructions against seismically induced oscillations by using special devices is generally accepted as an efficient technique. This method can be used for new structures as well as for the consolidation of existing ones.

Nowadays, it is applied in situations where classic methods do not succeed. Current design practice concentrate on increasing seismic protection by consolidation and strengthening, more than on trying to reduce and attenuate the effects of the seismic impact on the structures.

Such procedure needs a precise evaluation of the structural response to dynamic actions, like earthquakes, having the purpose to optimally calibrate the type of static consolidation.

This procedure, suitable for new structures design, becomes difficult to use in the case of existing buildings, especially for monumental buildings, because the evaluation of the seismic behaviour of an ambiguous structural typology is difficult to model, due to the vagueness of the static structural scheme, the mechanical behaviour of elements, etc. All these aspects leads to the conclusion that the dynamic analysis for these cases does not always work, especially if a post-elastic resistance is expected. That is why the analysis is limited to the supposition that the structure has a perfect linear elastic behaviour and neglects all cases of known nonlinearity (which take into account the structural geometry and inelastic behaviour of materials). Thus, the analysis must be limited to minor or medium intensity earthquakes, like those used in design codes, without any evaluation of the structure ductility reserve or the structure level of safety in the case of major earthquakes.

The known limits of this method are the following:

- the impossibility to approximate the seismic response of the structure, especially the dissipation modes by local plastic mechanisms, having as a consequence the impossibility to evaluate the damage level of the structure after the earthquake.
- the necessity of more or less substantially consolidations, which, in some cases, can lead to a complete modification of structural scheme, with not so good consequences for the architectural value of the edifice.

Based on these problems, the dominant is tendency to ececute “soft” interventions characterized by a reduced extent and using similarly techniques. This attitude was embraced by design codes which define two concepts of intervention in the case of monumental buildings: a more rigorous seismic ”improvement” and a seismic “adequacy”. These practices, widely spread due to practical and economical reasons, do not treat the typical problems of antiseismic safety. The technique based on seismic isolation is the newest method for antiseismic design criteria. This technique eliminates the limitations imposed by the classical procedure, trying to reduce the seismic energy in opposition with the attempt to increase the possibility of its dissipation. Therefore, it is required to position the devices in the structure’s key points, devices which will dissipate a fraction of the seismically induced energy, or, at least, this energy will not be totally transmitted to the structure [195].

This concept is not new, e.g. the protection of the electric installations fuses are disposed so as to disconnect the network from the general electric generator in the case of an accidental supercharges. The seismic load reduction devices for a building can be treated analogically. In the same manner, the mechanical shock absorption devices for vehicles has the purpose to reduce the induced vibrations and to avoid their transmission to the chassis. This is about the modification of the own vibration period of the system and about avoiding the resonance phenomenon.

As it will be further explained, the seismic isolation can be applied in two distinct approaches [195]:

- the one of the period T;
- the one of the capacity R.

The first one consists in the reduction of spectral accelerations obtained by the modification of natural frequency of oscillating at low values. The second one supposes that a fraction of the seismic energy is dissipated by dissipaters having as a consequence the reduction of the seismic forces, the maximum value is limited to the maximum capacity (R) of the devices.

The characteristics of the seismic isolation system recommend it for the seismic protection of monumental buildings when traditional methods are not usually suitable. It is enough to think about old masonry walls which do not have the necessary ductility as to overpass an earthquake, and even if they are consolidated it is difficult for them to achieve the desired level of insurance, or for the churches for which a rigorous consolidation would diminish the esthetical and architectural value, or structures the destination of which requires a continuous people flow that cannot be interrupted for the consolidation works. The advantages of seismic isolation can be of an economic nature when it limits the consolidation intervention works.

Therefore, we can get the advantage of being able to evaluate the seismic behaviour of the structure, supposing a perfect elastic linear behaviour, having the advantage of rapidity of calculus.

The two different conceptual approaches find their practical implementation by means of two techniques:

- the structural isolation technique;
- the energy dissipation technique.

The structural isolation technique introduces one or more discontinuities in the structure, disposed on its height and named isolation planes [110]. If this plane is situated between the infrastructure and the superstructure, the technique is named „base isolation".

2.2.2.2. The evolution of antiseismic design criteria

The first technical attempts to limit the effects of earthquakes were recorded at the beginning of the 20th century, period of the first design codes, which appeared as a consequence of disastrous telluric events.

These design codes generally provided constructive rules and wanted to limit the buildings height and mass. Very few design tools were available. These design codes being unrefined and approximate have paid the price in the earthquakes that followed.

Only after the Second World War there appeared design procedures based on rigorous methods for structural analysis and on the realistic evaluation of seismic forces. Even so, this approach proved insufficient until an advanced calculus technology appeared, because the calculus was very laborious.

These limitations were reflected in the evaluation of inertial forces, which were established by using simplified methods that supposed a modest seismic load. This choice was imposed by the necessity to make structural elastic analysis and relatively to the conventional value of the seismic action, which in many cases underestimates the real value of the seismic action. Other typical limitations were related to the impossibility to differentiate the distribution of seismic forces, function of the structural type, the structure destination or the foundation soil. In other words, the necessity to lead the structural design on each level (due to the impossibility to solve a structure with a great degree of statically indeterminacy) affected by much the design model.

Only in the last three-four decades, along with the improvement of design tools and the improvement of the knowledge about structural behaviour to seismic actions, an anti-seismic design philosophy based on the structural ductility concept was developed, understanding by this the capacity of the structural system to dissipate seismic energy by forming a plastic deformation mechanism.

Due to clear definition of the relation between the inertial forces and the deformation and ductility properties of the structural system, substantial improvements were made even for traditional approaches, where usually only an elastic analysis of the oscillating system was made.

All these recent developments for the calculus and design methodology are successively met by the design codes of the ‘70s. Normally, there are, also, limitations for this approach. These limitations can be seen in the difficulty to define a ductility level, the impossibility to ensure for a certain structural typology the adequate mode to form a dissipation mechanism and the necessity to evaluate damages. This lack negatively reflects under two aspects:

- the evaluation of the safety level of the structural system, which results in the difficult quantification of the effective structural ductility;
- the high cost of the building and consolidation of the structure in order to reach a high degree of seismic safety.

From the supposition that the seismic impact can be modeled as a flux of elastic waves, the study of a structure becomes the analysis of a vibrating system under a pulsatory excitation. In this case, the analytical instrument is the differential equation of the seismic motion, the integration of which is the main element in the study of the dynamic structural response.

There are two distinct methods for dynamical structural analysis which differ by the input and definition of the seismic action and by the adopted calculus procedure.

The first one is based on the direct integration of the equation of motion, established from a real or an artificial code earthquake, and the second procedure is based on a modal analysis based on a design spectra. Both methods are based on a series of hypotheses, which in practice represents the conventional methodology fundamentals of antiseismic design.

The first hypothesis refers to the fact that all points of the building base are in the same vibration phase. This is not a too drastic limitation for a multi storey building, but can become least viable in the case of buildings with large in-plane dimensions related to the foundation soil, like monumental buildings or large span bridges.

The most limiting hypothesis is the one that assumes, generally, the linear behaviour of the building. Thusall known nonlinearities are excluded, like geometry variation, plastic behaviour of the materials, etc. From this arises the need for analysis stint to the elastic domain for medium intensity earthquakes characterised by a recurrence period of a few decades.

From another point of view, we cannot admit for economical reasons that the structure behaves elastically after major seism motion with a recurrence period of centuries, understanding that, in this case, the structure will suffer irreversible damage, still being able to preserve sufficient stability as to ensure the inhabitants' life. The verification of a structure in the event of a major earthquake must ensure that the bearing structure should have the appropriate ductility in order to dissipate the energy that exceeds the elastic resources.

Unfortunately, because of the difficulty in evaluating the inelastic behaviour of the structure, the ductility control is easily solved by paying special attention during the design and construction to certain constructive details which ensure the plastic hinge formation – dissipative mechanisms which influence the seismic behaviour of the structure. It is obvious that this approach is more qualitative than quantitave, thus being far away from the engineering practice.

Besides the impossibility to control the dissipated energy by local mechanisms, the progressive degradation of resistance and material rigidity which influence the results, must be taken into account (i.e. phenomena like concrete and masonry degradation, cleavage of walls at intersections, the instability of the compressed reinforcement, the reinforcement pull-out, the local or global stability of slender elements, etc.). There are some structural typologies with core or strengthened parts which have insufficient capacity to dissipate energy by plastic deformation, but in any way these structure are favourable for limiting the excessive horizontal deformations and deteriorations of complementary works.

All in all, the usual approach in design implies the necessity to ensure sufficient strengthening of the structure in order to resist a design code seism. This strengthening strategy has a few flaws:

- the evaluation of post-elastic capacity is imprecise, and that is why any structural damage prevision is impossible;
- high costs during set-up and for consolidations, if damages appear; they are very expensive particularly for low ductility structures;
- it is usually inapplicable for monumental buildings, for which the resistance case cannot be modified;
- does not foresee in any way the possibility to limit the earthquake impact if the produced damage changes ad hoc the dynamic response of the structure.

The analysis of an antiseismic design method based on soil-structure interface modification by inseration of special devices can be appropriate. The purpose of this operation is to diminish the seismic forces effect by increasing the own period of the structure and in consequence to diminish spectral accelerations. This procedure is called seismic isolation and is the most advanced method for antiseismic protection until now. Thus into this way into this category fall all the actions which use dissipative devices.

2.2.2.3. The prerogatives of an antiseismic conformed structure

Each earthquake has underlined the downsides of the previous design methods and afterwards the need of redefining and re-evaluating with more accuracy the structural demands of constructions located in highly seismic areas.

An earthquake is not only a hazard for human lives but a great loss for the built heritage. Two similar earthquakes, in Japan 1985 (Tokyo) and Italy 1976 (Friuli), had different results due to different approaches for antiseismic design.

The constructions built in areas with a high seismic risk must have the following essential characteristics:

The principal resisting elements of the structure must be strong enough to undertake medium intensity earthquakes without damage for a period of recurrence of one-two decades, which represents the technical and economical operating period for a specified construction. This requires that the structure should remain elastic during a medium intensity earthquake.

All structural elements (primary or secondary) must have sufficient ductility in order to dissipate the energy induced by earthquake without forming a local or global failure mechanism. In the event of major earthquakes, it is un-economical to design the structure so as to remain elastic, so damage is accepted to primary and secondary elements, in order for these to form mechanisms for energy dissipation.

These prerogatives are relatively easy to meet by new constructions built with new and modern technologies and new design methods, but in the case of old constructions built by old techniques, it is much more difficult to comply with the ductility demands. On the other hand, even if by consolidation the elastic resistance can be improved, it is impossible to ensure the necessary ductility and a rational energy dissipating mechanism. It must be considered that this type of consolidations does not provide a rigorous adaptation to anti seismic demands, due to economical reasons (substantial interventions are needed in order to review the static scheme) and technical reasons.

Within the antiseismic protection strategy, there arises the problem of finding a satisfying solution for the protection of the vulnerable monuments or those of great artistic value. A solution can be found by means of seismic isolation, which can limitate the seismic impact on the structure.

2.2.2.4. The energetic approach of antiseismic isolation

In order to fully understand the basics of the isolation concept it is required to understand the energetic behaviour of a structure under seismic action.

We have underlined the possibility to dissipate energy by means of plastic mechanisms which give the ductility characteristics of the materials and represents a basic component of the anti-seismic conformation. All recent worldwide design standards are based on this concept and adopt this position, by paying great attention to the ductile character of a certain structural typology in the seismic design evaluation. The concept of global ductility derives from the notion of local ductility: in fact, the ductile global behaviour greatly depends on the presence into its composition of a series of elements with high local ductility. The sum of these local ductilities forms the global ductility.

It is possible to improve the ductility characteristics of a building by a conventional reduction of the seismic action based on the choice of inelastic spectra.

A sufficient ductility offers great plastic deformation capacity allowing it to dissipate a great quantity of energy thus significantly reducing the intensity of seismic inertial forces induced into the structure.

This approach needs a special attention due to the following aspects:

- the difficulty to define the quantity and to quantify the available ductility of a structural system;
- the strengthening effect of non structural elements;
- the effect of the own vibration period variation due to plastic deformation.

The last aspect can lead to a reduction of design seismic loading for all periods greater than the control period.

A design based on this philosophy of structural ductility leads to the optimisation of structural systems so that they can form a complete mechanism of plasticization with a high degree of ductility (one of these systems is based on the concept of "strong columns" and "weak beams", because this the formation of the plastic hinges is guided to the beam's extremities, thus ensuring the maximum possibility of energy dissipation , excluding in the same time the risk of premature failure of the construction which may occur in the same time with first the plastic hinges in columns).

Also, there is the delicate problem of plastic hinges propagation control in the structure due to relatively quick material degradation and even the control of plastic hinges formation process. It is difficult to define a parameter which is able to characterize univocally the ductility. In scientific literature, the ductility, global or local, is defined as the ratio of a deformation parameter and the elastic value for the same parameter.

It is very difficult to evaluate the ductility of a structural component, in which the transit from elastic to plastic cannot be neatly defined. Finally, in the case of a cyclic loading, it is not possible to discuss about a single ductility factor, the evaluation of material degradation being needed.

These limitations can be removed by an energetic approach of the problem, being possible to take into account both the ductility and the resistance of the structure. In fact, these two characteristic are held directly responsible for the capacity of absorbing seismic energy. This characteristic is defined by some authors as the "tenacity" of the system, by illustrating the fact that the ductility (local and global) and resistance effects cumulate due to the contribution of non-structural elements.

In order to set straight the basis of seismic isolation by an energetic approach, references can be made regarding a single degree of freedom system SODF for which the mathematical model is easier and more general.

The dynamic response of a SDOF can be modeled by the equation of motion: $m\ddot{x} + c\dot{x} + kx = -m\ddot{x}_g$ by multiplying with $\dot{x}$ and integrating the system with respect to time, an equation of energetic equilibrium is obtained:

$$\underbrace{\int m\ddot{x}\dot{x}dt}_{E_k(t)} + \underbrace{\int c\dot{x}^2 dt}_{E_x(t)} + \underbrace{\int kx\dot{x}dt}_{E_A(t)} = \underbrace{\int (-m\ddot{x}_g)\dot{x}dt}_{E_I(t)} \tag{1}$$

where $E_k(t)$ – kinetic energy of the system's masses; $E_x(t)$ –energy dissipated by viscous linear damping; $E_A(t)$ – elastic energy absorbed by the system. The sum of these internal energies must be equal in order to compensate the input energy $E_I(t)$, from the external system's excitation.

The elastic dissipation capacity is generally exceeded in the event of high intensity seismic actions, certain elements begin yielding, others fails, so that the linear-elastic term from the above equation will be replaced by a nonlinear term function of $f(x,\dot{x})$ which implies a hysteretic dissipation behaviour.

The energy absorbed by the system $E_A(t)$ – can be divided into:

- Elastic energy $E_e(t)$;
- Plastic energy $E_p(t)$.

More generally, the above equation can be written [214]:

$E_k(t) + E_x(t) + E_e(t) + E_p(t) = E_I(t)$, and, even more general, the equation can be written as an energetic balance of the total energy exchanged by the system: $E_x(t_f) + E_p(t_f) = E_I(t_f)$

The sum of energy dissipated by plastic mechanisms with hysteretic behaviour and the energy dissipated by viscous mechanisms must be equal to the induced energy to the system by exterior excitation.

Particularly if the internal forces do not exceed the elastic capacity E_p=0 all the seismic energy is dissipated by viscous mechanisms. If, on the contrary, E_x is insufficient then a hysteretic dissipation is needed and it can be considered that all the energy induced to the system is dissipated by plastic mechanisms. This case is considered to be viable for most consolidation cases:

$$E_p = E_I \tag{2}$$

The energetic aspect presents the advantage of treating the problem in terms of input – output, characterising the structural behaviour globally. The criteria that governs the anti-seismic protection strategy can be deduced by an equation of energetic balance. From this relation, three terms can be underlined:

- energy dissipated by viscous phenomena E_x. This energy can be increased by starting from lateral displacements.
- energy dissipated by hysteretic phenomena E_p. In order to increase the plastic dissipation capacity in needed to ensure a ductile character for the elements and limit, as much as possible, the mechanic degradation phenomena. A structure with high E_p is a structure with high global ductility and a large number of ductile joints or devices with stable hysteretic behaviour.
- induced seismic energy E_I. It can be influenced by interventions on the structure and special interventions on its deformability.

The third strategy is called seismic isolation. In the literature under this name fall all the operations that reduce the seismic load without the modification capacity of the structure's capacity to dissipate energy. This strategy was especially studied for the seismic protection of existing buildings for which the strengthening operations were difficult. These dissipating devices can be set up without the essential modification of the resistance of the structural resisting case. Essentially, the dissipating devices are set up into the key points of the structure modifying its seismic response but in the same time remaining independent. In a more particular case of pure seismic isolation it is tried to reduce the induced seismic energy without the introduction of dissipative devices into the structure.

So, for "structural control", from the mathematical point of view, into the equation of energetic balance a new force controlled term must be introduced, that is u. The energy dissipated by this new term is $\int u\dot{x}dt$. This comes, by compensation, to reduce the dissipated energy by yielding and elements fracture. These devices are set up in areas with easy access, in order to easily maintain and eventually replace them. As underlined before, the vibration control produced by exterior sources is a more suitable solution than the conventional methods.

The vibration control of a structure can be achieved traditionally by modifying the rigidity, the masses, the damping or the conformation. New approaches propose new control methods by introducing new forces into the system.

These modern methods can be classified as follows:

- Active control: when certain devices supplied by an external source induce, after a prescribed scheme, into the system, certain forces. These forces can be used in order to introduce into and dissipate energy from the system. The actuators response depends on certain physical sensors which record the structural response. This type of control is popular enough for new structures, but it is not suitable for the existing structural consolidation methods;

- Passive control: no external energy sources are needed, and generally the forces that occur are translated into structure's motion response;
- Hybrid control: it is usually defined as a system composed of active and passive elements;
- Semi-active control: is an active control, with the difference that less energy is required than for conventional active systems. Generally, semi-active systems do not induce mechanical energy into the system, but ensure stability. This type of systems are mostly seen as passive controllable systems.

2.2.2.5. The demands of isolated structural systems

The above paragraphs discussed the demands and performance of a non isolated system, referring to various systems which modify the seismic response and ductility.

For the purpose of a superior performance of a building designed in accordance to traditional methods, an isolated structure must comply with the following demands [195]:

- the improvement of general safety conditions, in order to reduce to the minimum the structural damage in the event of a major earthquake;
- the reduction of seismic forces considered by the standard in the structural design, relative to a medium intensity earthquake;
- the possibility to adapt the constructive systems with the help of low ductility materials, which are characterised by a small reserve of plastic deformability;
- seismic adaptation of existing buildings.

Even if the first requirement is not fulfilled by traditional strategies, the advantage of isolated structures can be seen for the structures of maximum importance for which no damage is allowed. For monumental buildings that are part of the artistic heritage and for buildings with intrinsic economical value, the seismic base isolation is fully justified. On the other hand, even in a traditional approach the base isolation system shows an important advantage, that is to allow the use of reduced seismic forces for the structural design, as presented in the second requirement.

Requirements c) permits the rational and efficient use of non-ductile but cheap materials, with great economical benefits.

Requirements d) underlines an important and actual problem for all the engineering community, namely the preservation of historical buildings and monuments. Seismic adaption using base isolation methods does not necessarily require a major intervention on the existing resistance structure with the obvious benefits of not modifying or altering in any way the architectural value of the edifice. Thus, the seismic base isolation is not only considered a particular method to save certain monumental edifices, but also a viable economical alternative to traditional methods that use conventional materials and to successive maintenance operations. In this regard, we can underline that seismic base isolation is a very good answer for the consolidation of a wide range of structures for which the application of traditional methods would be impossible.

The isolation technique, in the case of existing buildings, must ensure, in contrast with traditional methods, the substantial reduction of seismic loads by reducing seismic energy, thus opening new perspectives on preserving built heritage.

2.2.2.6. Design strategies for isolated systems

As presented in the previous chapters, the basic working concept of this systems is the following [110]:

- The modifies the seismic response of the structure;
- It increases the quantity of dissipated energy;

Starting from this, the isolating systems can be classified into the following distinct categories:

- *Base isolated structural systems.* The purpose of this type of systems is the reduction of seismic loading by means of increasing the own vibration period. The most used devices are elastomeric devices, lead based devices.
- *Partial base isolated system.* Into this category fall the systems which insolate only a certain part of the structure. A typical example of applying this system can be found in the field of bridges where the support elements are isolated.

Generalised passive control devices: under this name can be found a series of applicative devices which dissipate energy by hysteretic or viscous means. Oleodynamic devices also fall into this category.

The following chart (see Figure 2-69) presenteds a simplified classification of modern structural control systems.

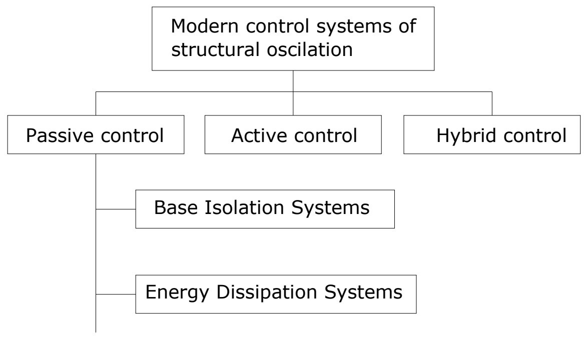


Figure 2-69 Modern control systems

It is important to identify some possible types of isolation and the isolator or dissipator’s position in the structure. After a primary analysis, it can be observed that these devices are placed in the points corresponding to the maximum values for displacements generated by the inertial horizontal actions.

Seismic isolation systems can be divided into three big conceptual categories according to their working principles [205]:

- Period elongation systems (PE) – the reduction of the seismic forces is mainly done by means of horizontal flexible elastic supports.
- Force barrier systems (FB) have a plastic rigid behaviour or even elastic nonlinear with zero or very low consolidation. The systems are characterised by a well defined step force which prevents the transmission of forces greater than it to the superstructure.

- Period elongation / Energy dissipation systems (PE/ED) – The reduction of seismic forces is achieved by the increasing of the own period as well as by energy dissipation. They are made of horizontal deformable supports which act as a nonlinear hysteretic system with high energy dissipation capacity.

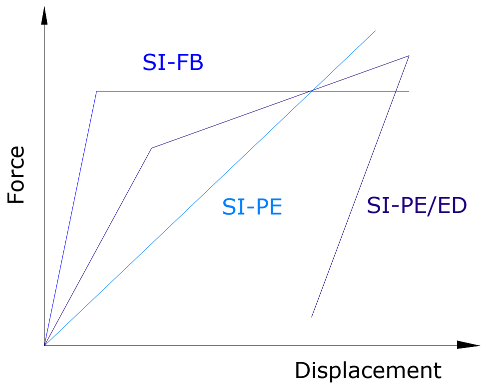


Figure 2-70 Conceptual behaviour of seismic isolation systems

In order to select the right system, a series of factors have to be taken into account:

- *The level of the own frequencies of earthquakes which are expected to occur on the given location.* If there are expected components of great amplitudes in the domain of low frequencies, the base isolation and own period increasing systems have to be eliminated, the energy dissipating systems being a better option.
- *The building type and structural system.* Normally, the base isolation applies to rigid low and medium rise buildings whereas, the energy dissipaters are recommended for flexible frame structures. Specific structural configurations can require ad hoc decisions.
- *The necessity of content protection* from high frequencies of vibration or in order to avoid people's panic inside an overcrowded building. In these cases, the seismic isolation is the best suited solution.

In case of consolidation supplementary difficulties arise because of the real on-site situation (geometrical limitation, structural system type, actual structural resistance, etc) which reduce the practical possibilities of the system set up.

General design criteria are set to obtain a correct behaviour of the overall structural system:

- to build structural networks underneath and over the isolation system of a building;
- to install ultimate displacement limiting systems;
- the devices to be easily inspectable and replacable;
- to ensure the compatibility with vertical service loadings (vertical rigidity);
- to prevent torsion effects;
- to ensure the compatibility between structural and non-structural connections with design displacements;

By contrast, the demands of designing a new building and the demands of seismic rehabilitation of buildings are the following:

- choice of desired performance level;

- defining the expected behaviour of the building in the event of a design code seism in terms of damage limitations;
- seismic risk: determination of seismic motion and other location related risks;

The construction characteristics:

- Basic characteristics determination of the building and resistance capacity of the existing building at an earthquake;
- Rehabilitation methods: the choice of simplified or systematic method;
- Rehabilitation strategies: the choice of the basic strategy, e.g. supplementary elements able to undertake lateral loads;
- General and design analysis: the specification of force type actions and deformations for which the given components of a structure and the setting of the minimum interconnection criterion for structural elements must be evaluated;
- Analysis and design procedures: for systematic rehabilitation approximations, the selection between linear static, linear dynamic, nonlinear static and nonlinear dynamic.

2.2.2.7. Reduction devices

These devices have the purpose to reduce the negative effects introduced into the structure by the action of the seismic force. The fundamental principle provides that this type of isolator devices permit the ground motion without transmitting it to the structure. In the real situation, there is no need of contact between the building and the supporting base. The functioning principle is illustrated in the Figure 2-71. Also the ways that a rigid and a flexible structure respond to seismic action are presented in Figure 2-72.

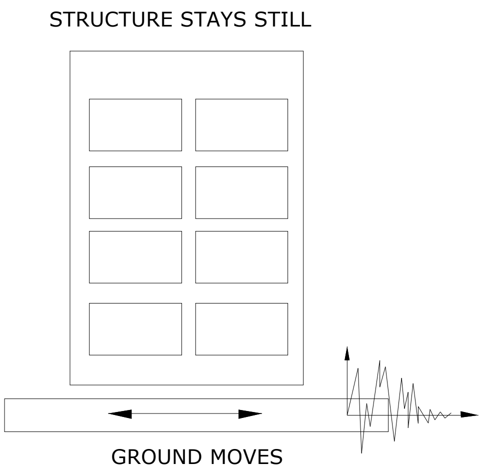


Figure 2-71 Principles of based isolation systems [110]

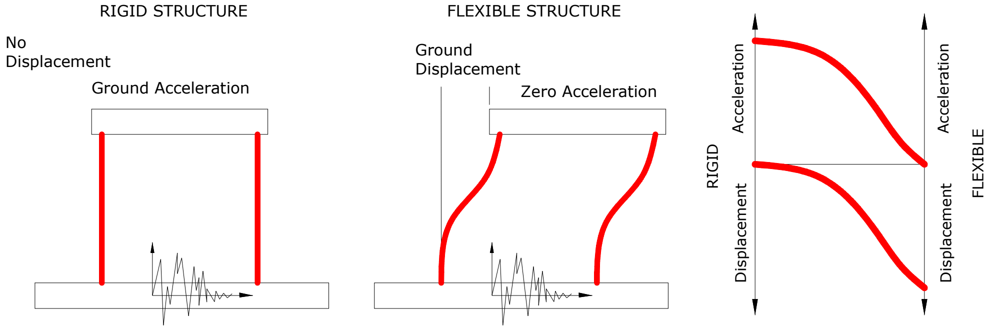


Figure 2-72 The comparison between the behaviour of a rigid and a flexible structure [110]

Over the time, many types of systems and isolating devices have been proposed. Many of them were put into practice, others were just proposals and others were more in the state of concept and impossible to achive in practice. Further on, the principal types of systems available on the market will be presented.

Sliding Systems – sliding or friction isolators

This type of systems has a simple setup based on a clear and easy-to-use concept. Besides this, the working principle is backed up by a facile theoretical implementation. One layer with a certain friction coefficient will limit the acceleration and the forces transmitted into the system (building) at a prescribed value, equal to the ratio of the friction coefficient and the weight of the building.

A pure system does not have the displacements locked and they do not prevent the comeback of the structure to the state before the seismic action. This flaw can be removed by the combination with other devices which are able to exert comeback forces or by using spherical sliding surfaces (see Figure 2-73).

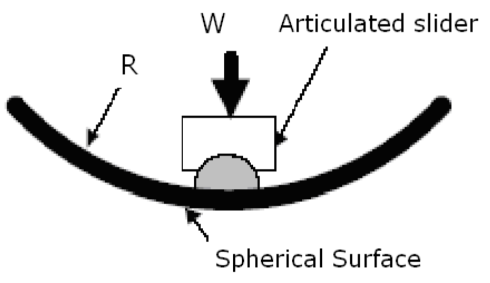


Figure 2-73 Scheme of sliding devices [110]

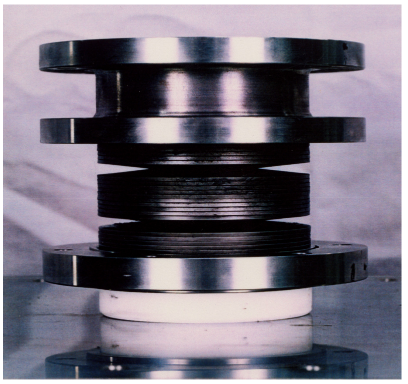


Figure 2-74 Scheme of friction damper

Elastomeric supports (rubber)

These devices are made of multiple layers from thin natural or artificial rubber tied together by metal plates (see Figure 2-75). The metal plates prevent the swelling and excessive deformation of the device under vertical loads which can lead to stability loss for these elements. Under vertical loads, they show reduced deformations, while, in the case of horizontal loading, they are very flexible. Plane elastomeric supports assure flexibility but not enough damping and they will deflect under service loads. In order to remove this flaw, a lead core is inserted in these devices (see Figure 2-75).

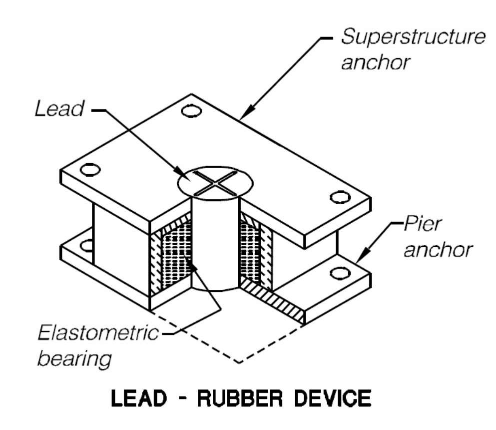


Figure 2-75 Scheme of elastomeric supports [216]

Springs

They are specific devices based on metallic springs (see Figure 2-76). This family of devices is not widely used and they are generally used for the isolation of mechanical machines. The main disadvantage of this type of isolators is the increased flexibility, horizontal and vertical. The vertical flexibility leads to unfavourable dynamic responses. The decreased damping and excessive displacement under service loads make the independent use of this kind of insulators improper.

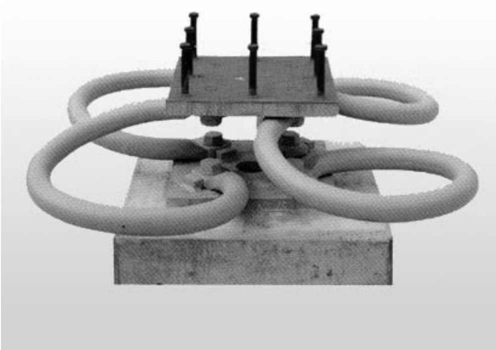


Figure 2-76 Steel damper [168]

Roller and spherical supports - isolators though rolling

They include cylinder rollers as well as spherical devices. As springs, they are mostly used in mechanical engineering applications. Depending on the support material, they can offer a good resistance to displacements and sufficient capacity for service loadings.

Flexible storeys and short pendulums

The flexibility is ensured by shot pendulum columns (pinned at both ends) which allow displacements or transform the storey in a "soft" storey. This system offers flexibility, but does not ensure damping and resistance under service loads, and that is why they are used together with other systems.

2.2.2.8. Dissipating devices

Some of the devices presented before offer flexibility but does not offer sufficient damping and therefore supplementary devices are introduced into the system:

- Viscous dissipaters (oil dissipaters) – these devices offer a good damping but they do not have any resistance to service loads. They do not have elastic rigidity and therefore they introduce into system only a small amount of energy;
- Devices based on steel yielding – set in such a way as to yield to various types of efforts, as bending, torsion, etc. They offer rigidity as well as a good damping and dissipation (see Figure 2-77). Devices based on lead yielding act to shear and offer rigidity and damping.

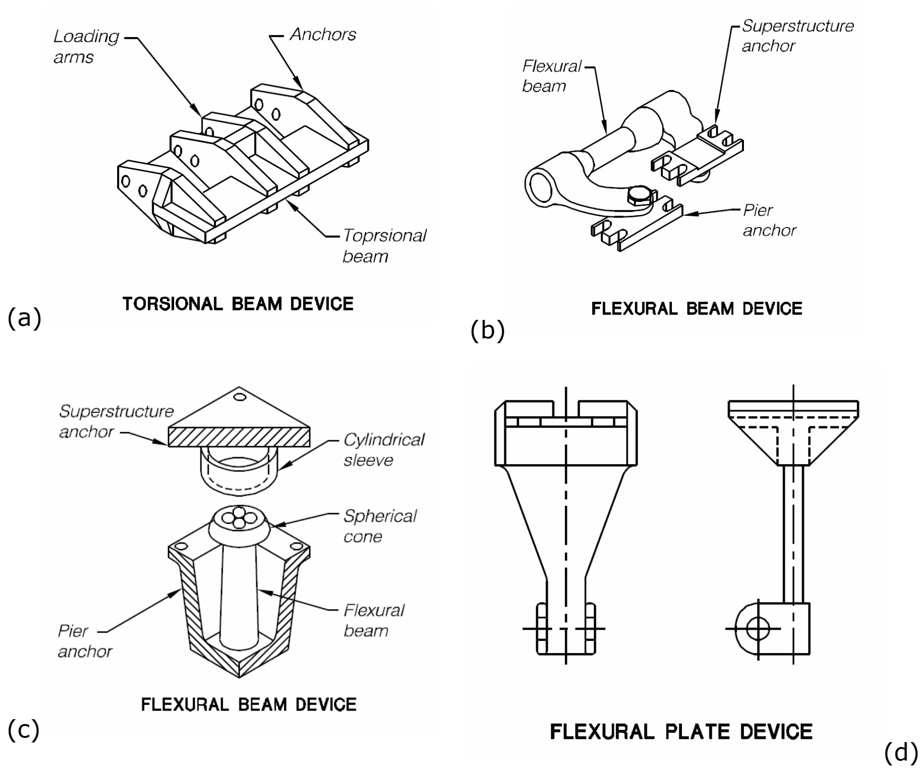


Figure 2-77 Steel dissipating devices (a) torsional beam device; (b)(c) flexural beam device; (d) flexural plate device [216]

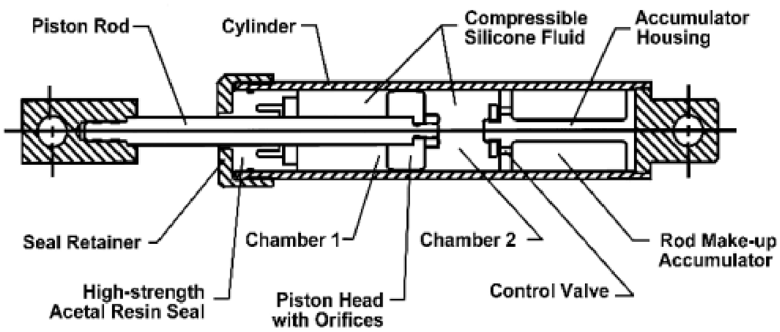


Figure 2-78 Fluid damper [203]

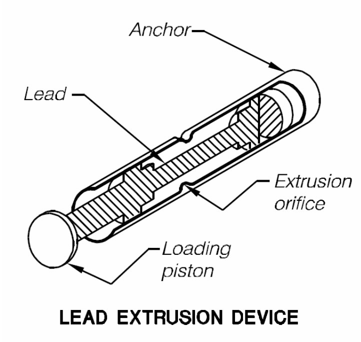
(a)  (b) 

Figure 2-79 Lead damper (a) and lead extrusion [168] (b) devices [216]

Devices based on extruded lead (see Figure 2-79), in which lead is forced to pass through an orifice , they add rigidity and damping to the system.

All these devices, except for the viscous dissipaters, are displacement dependents and therefore they offer maximum force for a maximum displacement. The viscous dissipaters are velocity dependent being able to offer a maximum force to zero displacement, therefore they can be more appropriate than other devices in the case of application on rigid buildings (e.g. masonry buildings).

A more accurate classification can be found in the Table 2-3.

Table 2-3. Advanced devices for structural control

Isolators	Laminated Rubber Bearings	Natural Rubber Bearings
		Lead Rubber Bearings
		High-Damping Rubber Bearings
	Slide Bearings	Elastic Slide Beraings
		Rigid Slide Beraings

Advances devices for structural control (continued)			
Dampers	Hysteretic-type Dampers	Steel Dampers	Bar Type
			Loop Type
			Portal Type
			Plate Type
			Ring Type
		Lead Dampers	
		Friction Dampers	
	Velocity-type Dampers	Oil Dampers	
		Viscous Dampers	

Further on are presented the hysteretic curves of different types of devices.

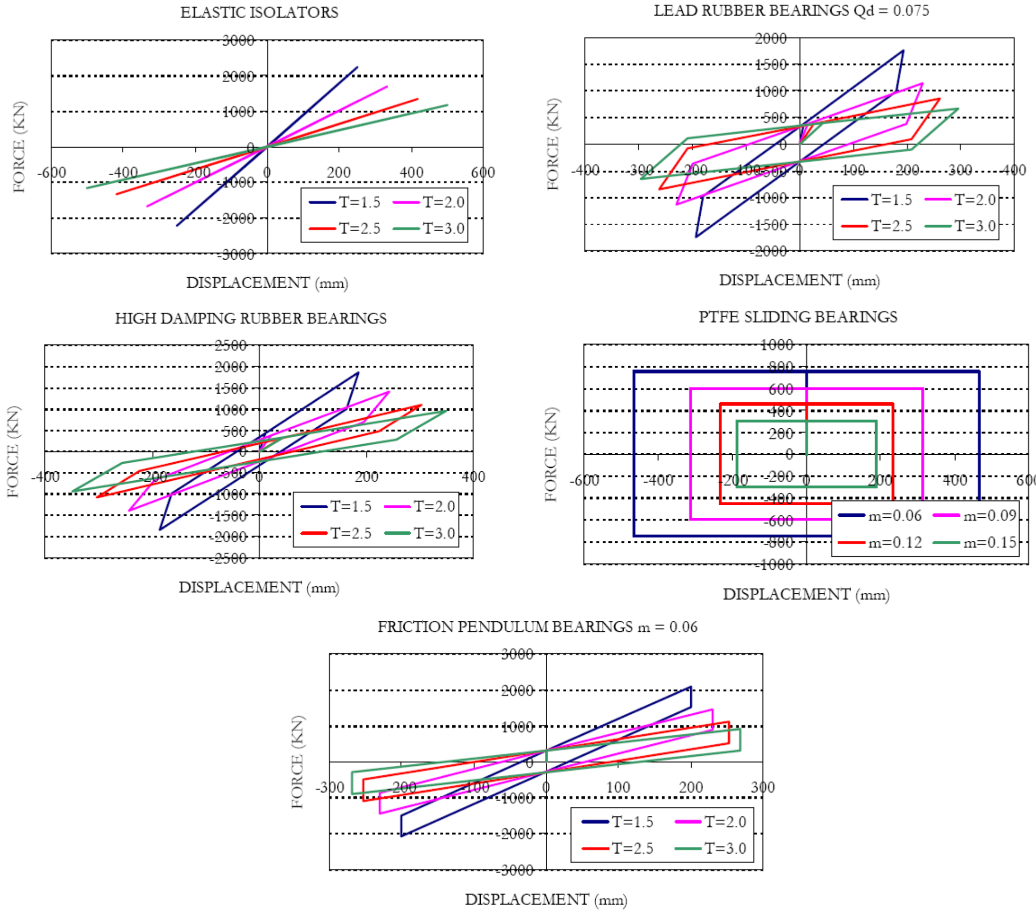


Figure 2-80. Typical hysteretic loops for different types of devices [110]

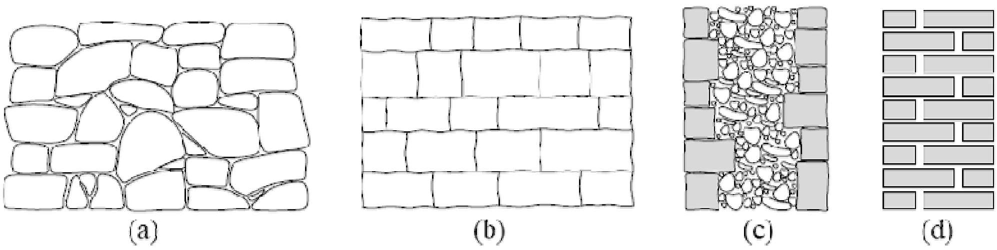
2.3. SEISMIC RETROFITTING ON MASONRY BUILDINGS

2.3.1. General features of the masonry behaviour

The main problem of masonry is that it is a non-homogeneous material composed of two materials with different mechanical properties. The equivalence with a homogenous material is not easy to be done because of the dependency of masonry on several factors: properties of the materials, typology of masonry, quality of workmanship etc.

One of the main problems is the great vulnerability of the masonry to earthquake due to the inherent physical properties like the lack of resistance (small tensile resistance), small deformability and low ductility, having a sudden and brittle failure. On the other hand, the small ratio between the resistance and the own weight of the material (the masonry elements are massive with great mass and rigidity) attract high inertia forces.

The great variability of the masonry typology can be a serious inconvenience for the establishment of a concise methodology that should describe the characteristics of masonry behaviour in terms of mechanical properties. In different parts of the world and in different historical periods masonry has known a wide application from stone elements, independent or linked with earth based material to clay brick with or without mortar. Nowadays the brick unit used for masonry elements can be classified as: solid, perforated unit, hollow unit, cellular unit, horizontally perforated unit etc. The mortar can be classified as: general purpose mortar, thin layer mortar and lightweight mortar. In the following figure (Figure 2-81) some of the different types of masonry elements may be observed. This thesis will focus on clay brick with the remark that the general principles presented remain the same for all the other typologies, too.



(a) rubble masonry and (b) coursed ashlars masonry, and (c, d) possible cross sections

Figure 2-81 Examples of different kinds of stone masonry, [122]

From the following scheme (Figure 2-82) which shows the failure modes of a masonry structure when an earthquake occurrs, it may be observed that, due to the direction of the loads, the masonry panel shows different behaviours and different failure modes. A global analysis procedure (i.e. ultimate limit state), including the main failure modes – collapse mechanisms and the evaluation of the bearing capacity, is presented in detail in this thesis.

From this point of view the masonry panel failure modes may be divided in two categories:

- In-plane behaviour;
- Out-of-plane behaviour.

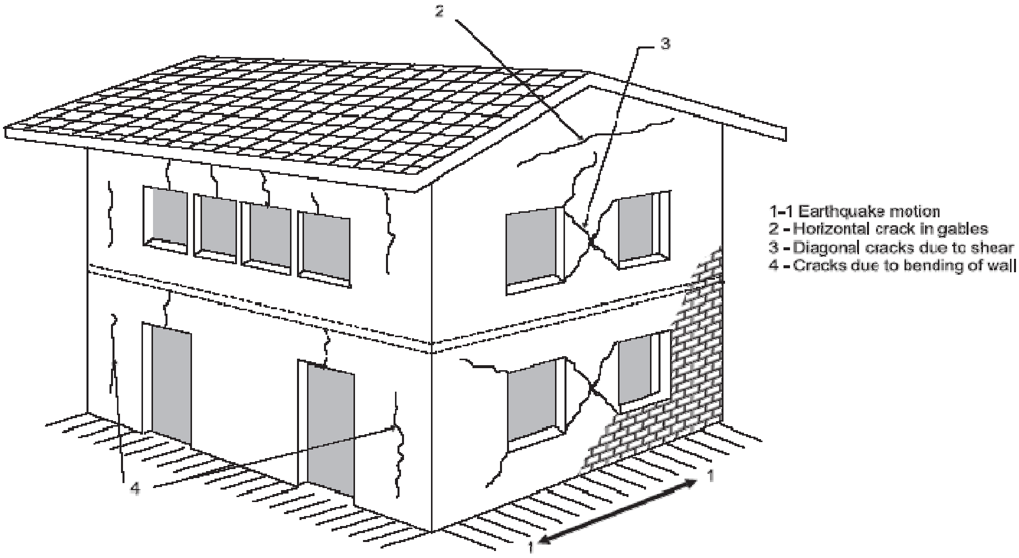
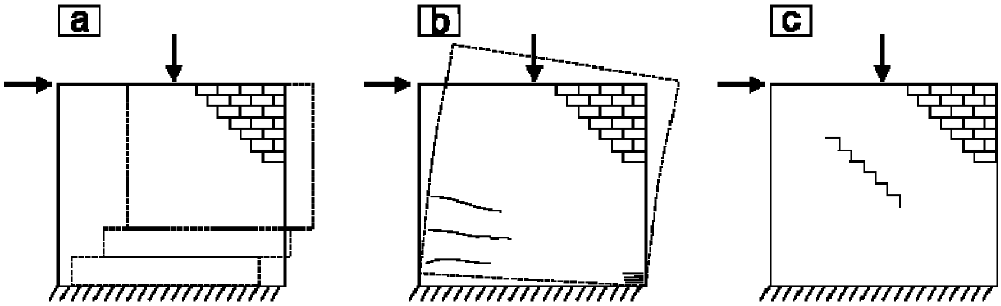


Figure 2-82 Masonry structure behaviour under earthquake action

Out-of plane failure occurs due to excessive horizontal forces perpendicular to the wall plane. This failure mode is observed when the diaphragms of the building are not effective and the wall acts like a standing cantilever with poor connection at the top. Long, tall and slender walls are very vulnerable. In massive historical buildings the self-weight of the wall generates important inertial forces in the event of earthquake, causing the wall panel to fail out of plane. This failure mode makes in case of masonry parts of the building to collapse. In order to avoid out-of-plane failure, there must be ensure effective diaphragm action at the floor and roof levels, and also a good connection at the corners between transversal and longitudinal walls.

In-plane failure of masonry walls occurs when the diaphragms are effective in transmitting horizontal forces to the walls placed parallel to the horizontal force. In this case, the walls may fail in-plane. As in-plane failure only occurs in the well configured buildings, with good 3D interaction between elements, in this case, the strengthening of the walls has to be the main focus of the rehabilitation, so as to resist horizontal forces.

Because out-of-plane failure is clearly related to bad 3D configuration, the rehabilitation techniques of masonry against in-plane failure are the main focus of this thesis. Anyway it is very important to mention again that out-of-plane failure mode is the one that causes parts to collapse, but by improving 3D configuration (conformation measures), rather than strengthening the masonry wall, the probability of occurrence of this failure mode can be restricted. This thesis will focus on describing the behaviour and the possibility to model the in-plane behaviour. When subjected to in-plane loads, a masonry panel can fail in one of the following ways (Figure 2-83), depending on the wall geometry and load conditions:



Sliding failure (a) Flexural failure (b) Shear failure (c)

Figure 2-83 In-plane failure modes of shear panels [131]

The typical in-plane failure modes for a masonry wall with openings are shown in Figure 2-84:

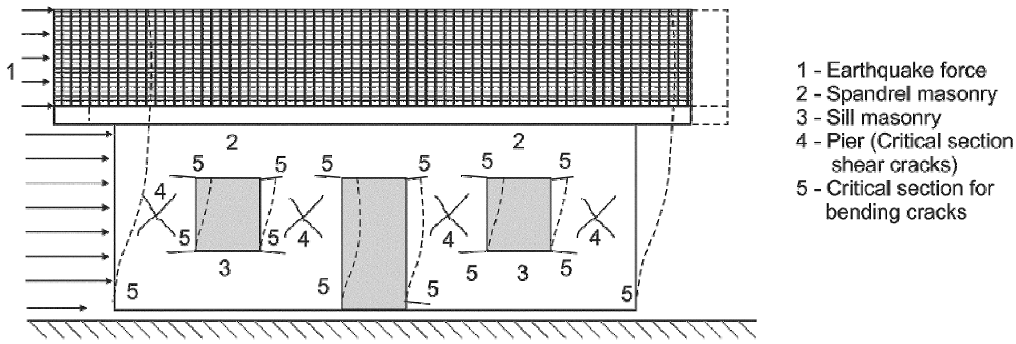


Figure 2-84 Critical failure modes in a masonry wall with openings, [102]

Walls subjected in-plane loads may fail in one of the three ways [102]:

- *Sliding failure* is defined as the horizontal movement of entire parts of the wall on a single brick layer, vapours barrier or mortar bed. It usually occurs in one of the lowest mortar beds of the wall in cases of wall with aspect ratio h/b lower than 1, and small level of vertical load. It is a non-ductile failure mode in the event of earthquake.
- *Flexural failure or rocking*, when the wall behaves as a vertical cantilever under lateral bending and, either cracking in the masonry tension zone (opening of bed joints) or crushing at the wall toe. It occurs in slender walls with aspect ration h/b bigger than 2. This failure mode usually involves large inelastic deformations without reduction of bearing capacity.
- *Shear failure* is characterized by a critical combination of principal tensile and compressive stresses as a result of applying combined shear and compression, and leads to typical diagonal cracks. In practice, two types of shear cracking can be observed, joint cracking by local sliding along the bed joint and diagonal cracking associated with cracks running through the bricks as well as the joints. It is the most common failure mode in case of masonry walls and unfortunately is a non-ductile one.

All these failure types should be considered when we try to determine the element resistance for the design or checking of a structure. The design codes for masonry offer for each failure type precise formulae for resistance associated with a failure mode. Also, the intervention strategy should select retrofitting techniques that are effective against the most probable failure mode to occur.

2.3.1.1. Rehabilitation of masonry buildings

Rehabilitation techniques are also numerous and they address a large variety of masonry typologies. Some of the techniques were developed for a specific typology of masonry; others can find general application in most masonry configurations. Depending on the damage level and the purpose of the rehabilitation works the intervention can be classified as:

Reparation involves only cosmetic repair of the wall, or of the finishing without any improving of the structural behaviour of the masonry. This approach has negative aspects like covering visible faults (e.g. cracks), thus living the impression of no problems. Sometimes after removing the surface reparations some major damage becomes visible in the bearing elements hidden beforehand and hindering the establishing of the real degree of structural degradation.

The *restoration* implies the remedy of existing damage in a masonry element. The goal is to restore the previous state of the masonry in terms of structural performance (i.e. strength and stiffness). This approach is generally applied to damaged masonry, judging that the initial behaviour of the wall was satisfactory under earthquake action. The condition which causes the damage can be: bad maintenance, water penetration, accident, explosion, settlement of supports or even earthquake.

When it is supposed that a masonry element would not perform well under expected loads (e.g. earthquakes), *strengthening* must be applied. Strengthening is mostly executed together with restoration after damage occurrs, for instance after damaging earthquakes.

If masonry walls have to be rehabilitated, the decision has to be taken related to the goal of the structural intervention. When the strategy is drawn, it should be considered that cosmetic reparations do not improve structural performance, while restoration and strengthening do.

The rehabilitation works have to start from the assessment of the entire structure. First, in order to improve the behaviour of load bearing masonry elements, some complementary measures, like the assurance of the rigid diaphragms, the consolidation of the foundations and the roof, the general antiseismic conformation measures, have to be taken, otherwise the retrofitting work can be non-efficient.

Following the Italian experience of recent earthquakes, failure modes of masonry structure were divided in two distinct categories [125]: a first stage failure defined as a local one, caused by the out-of-plane failure of walls, insufficient anchoring, tying or defficient diaphragm, generally because of the lack of antiseismic conformation. Because they don't lead to global collapse mechanisms, these failure modes can be studied on sub-models, without the complete modeling of the building; the second stage failure mechanisms are related to the entire building. Modeling of the whole building is needed for the assessment of these failure models.

The target of rehabilitation works, in the case of seismic rehabilitation, usually is to ensure that the structure should withstand a given earthquake level. In principle, this target can be achieved by increasing the strength and sometimes the ductility of connections between building elements in order to ensure effective 3D interraction; by increasing the floor diaphragm strength and stiffness in order to ensure uniform transmission of horizontal forces to vertical elements; by increasing the load bearing capacity (strength) of the vertical structural elements; by increasing the ductility of bearing elements; by enhancing energy dissipation; by modifying the stiffness, and hence changing the period of vibration, in order to reduce the earthquake's input.

Usually, by any intervention, a number of the above parameters will be affected. It is very improbable that, for example, the stiffness of a component can be changed without too much affecting the, or vice versa.

Traditional masonry rehabilitation techniques for vertical elements usually have the main goal of increasing the strength of elements or connections. However, it is important to remember that the intervention should improve the ductility and energy dissipation of the structure. It is believed that in the case of masonry the increase of deformation capacity is one of the most appropriate approaches.

2.3.1.2. Strengthening masonry shear walls

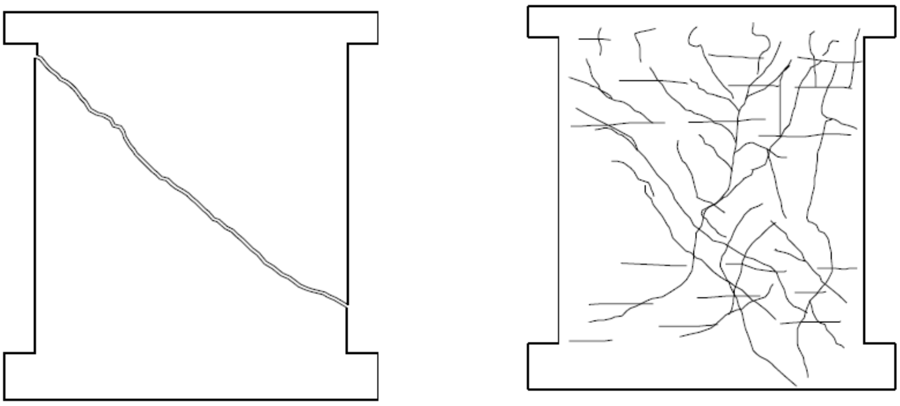
In order to select a rehabilitation technique, it is very important for the designer to know which is the expected failure mode of the masonry element. Increasing the strength implies strengthening the locations or parts which initiate this failure mode. By applying a strengthening technique that prohibits a certain failure mode, the designer forces the wall to fail in a different mode. So it is important to emphasise that in most of the cases not only the strength of the wall increases, but the mode of failure may also change. The failure modes of the masonry walls subjected in shear are detailed in 2.3.1.

As described above, the goals of in-plane wall strengthening directly derive from the target failure modes which are to be avoided or delayed. Generally, the retrofitting techniques address all or some of the failure modes. A summary table (Table 2-5) is also presented, noting which rehabilitation technique is effective against which failure mode. Depending on the aspect ratio of the walls a characteristic failure mode is most probably to exceed, as in Chapter 2.3.1. In the case of shear walls, some approaches are recommended.

If the aspect ratio of the wall is small (i.e. the height of the wall is small compared to the length), the wall will always fail in shear, and rocking behaviour cannot be obtained. Then the goal of the strengthening is to increase shear capacity and add ductility and energy dissipation. In order to avoid the collapse of the structure, the cracking should be distributed on as large an area as possible. Failure involving a significant part of the wall is usually superior to a localized failure in two ways, because it mobilizes all its strength, by activating all reserves (Figure 2-85). If the cracks are evenly distributed, the wall will withstand more overall deformation, and naturally supply more ductility and energy dissipation. Therefore failure modes that involve large parts of the wall are to be wished for in the event of an earthquake.

If the aspect ratio is large (i.e. the wall is very high compared to its length), rocking failure will occur and the strengthening solutions should focus on increasing the toes capacity against crushing on protecting the uplifted side against tensile stresses and on adding some shear strengthening in the middle part. Rocking is an advantageous failure mode, as it provides large deformation capacity and self-centering (i.e. at the end of the shaking remnant displacement is 0). Anyway, strengthening the uplifted side against tensile stresses has to be made jointly with the foundation proper anchorage assurance.

The walls with approximate aspect ratio of 1:1, as the target of the strengthening should increase both the shear capacity and the in-plane bending capacity and try to distribute the cracking on as large an area as possible. Many researches suggested [186] that strengthening should be so that shear cracking be delayed, allowing for the development of horizontal flexural cracks.



(a) (b)

Figure 2-85. (a) Brittle-shear failure and (b) ductile shear failure of wall panel [220]

If the initial state of the wall is damaged (e.g. the presence of cracks that greatly reduces the shear and tensile strength) the restoring or improvement of the bond between the cracked parts is needed. This can be achieved through *injection techniques* (e.g. using epoxy resin), with the aim to restore the bond between the two surfaces and to provide shear and tensile strength. Also *bandaging techniques* (e.g. by FRP mesh) circumvent the original/damaged stress path, and supply an alternative path for stress transmission. Both methods aim at recovering the shear and tensile strength over the cracks, and show improvement in the case of sliding shear and diagonal tensile failure modes.

It can be conclude that any retrofitting technique should be applied after the local repair of cracks by using either injection, or bandaging techniques.

In the case of bending capacity improvement, the confining of masonry plays the key role, not only by offering a superior compressive strength, but by providing ductility. As was stated above, the retrofitting technique sometimes changes the failure mechanism, so, in most cases, even if the initial failure mode was pure shear, after retrofit the failure mode is a combination between shear and bending and in this case the confinement effect is positive.

A very comprehensive repair guide can be found in FEMA 308 "Repair of earthquake damaged concrete and masonry wall buildings" prepared by ATC – The Applied Technology Council. This document divides the repairs of earthquake-damaged masonry wall buildings into three generic categories: cosmetic repairs, structural repairs and structural enhancements, similar with to 2.3.1.1. This document recommends for URM cosmetic repairs as: surface coating and repainting; structural repair: crack injection with grout and structural enhancement: concrete overlay and composite fibres, offering for each techniques a detailed description, materials and equipments used and execution and quality assurance, indicating the limitations and presenting relevant references [17].

2.3.1.3. Strenghtening of masonry columns and joints

Columns

The literature on jacketing RC columns is extensive, but not many papers refer to the strengthening of masonry columns. In earthquake loading situations the main problem of columns is related to flexural capacity and ductility, and in the case of short columns, insufficient shear strength might also be a problem [186].

The aim of columns strengthening is to ensure that shear strength is sufficient as to allow the column to develop a more ductile failure mechanism (i.e. flexural failure).

The increase of shear strength is achieved by the confinement of the column in the region of the potential plastic hinge. Passive confinement can be achieved by external jacketing. Jacketing can be made with steel or advanced composite material (i.e. wrapping), over the prepared and cleaned surface of the column. Circular columns only need surface preparation, but the cross-section of rectangular columns are usually changed into oval or circular by external concreting. This is necessary for the jacket to develop radial tension stresses [186]. For active confinement, the jacketed column is pressure-grouted either by mechanical prestressing or by heat applying.

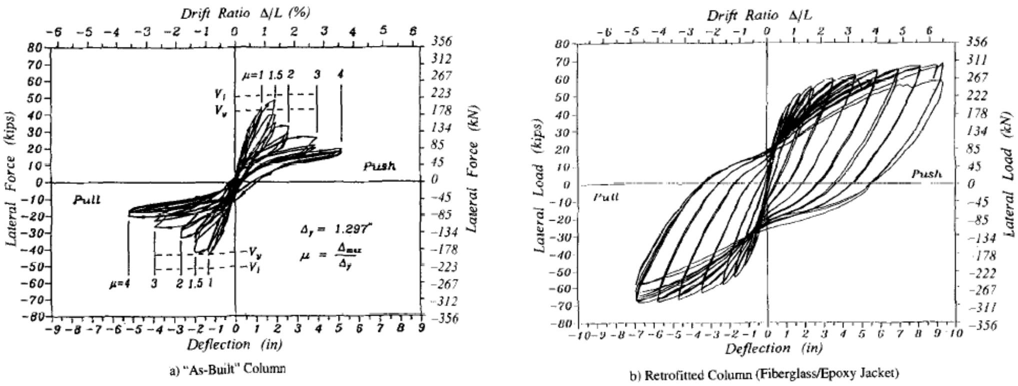


Figure 2-86. Force-displacement response of flexure dominated circular column. "As-built" and retrofitted with fiberglass/epoxy jacket [186]

Most of the principles used in FRP or steel jacketing of RC columns should be applicable to masonry columns. The main difference between strengthening of URM and RC columns is that the tensile and shear strength of masonry is smaller than the one of RC.

Providing extra longitudinal reinforcement, either by steel bars or by longitudinal fibers, is not always advantageous because it increases the flexural strength of the column, shear sudden failure becoming possible. The bending strength of columns rarely have a significant influence on the capacity of the structure, because in-plane walls are much stiffer and attract most of the load [80]. However, the displacement capacity of the columns is important so that they could follow the deformation of the structure.

Joints

The proper jointing of structural elements have to be consider as mandatory pre-requisite of good earthquake performance and is a first priority in order to insure a good 3D assembly. If a smooth transition of forces between the elements is not ensured (e.g. slab-to-wall; wall-to-wall; slab-to-column, wall-to-foundation) the retrofitting of load bearing elements will be inefficient. [125]. This thesis will not insist on the rehabilitation of joint typologies, but will underline their importance. Even if the floors are upgraded so as to behave as rigid diaphragms, the anchorage with existing walls is essentials. Moreover, the intersection of walls or between an existing walls a new wall must be provided with proper jointing. Traditionally, this can be achieved through RC beams or columns that conlucrate through connecting ties of steel [8].

Spectacular increases of the wall strength were reported by using FRP strengthening (see Chapter 2.3.3), which it is highly dependent on the anchorage of the FRP to the supporting elements of the walls. In reality, these strong elements are not present and rarely can be executed on site, or they only can be executed by hard work, by replacing or inserting new elements made of steel or concrete. Obtaining an increase in strength of 10 times is impossible without a perfect anchorage at the top and the bottom of the wall so as to prevent rocking and base sliding. The weak foundations are not to be neglected, as they will drastically influence the optimistic results.

If the top and the bottom are not reinforced (by, for example, steel dowels and FRP anchoring), the strength is limited by sliding shear strength of the wall at the base. Without the top and the bottom strengthening and anchoring, the strength increase obtained has been at maximum 120 % [80].

In conclusion experimental results show in most of case the "optimal case scenario" and can not be used as reliable results for a real retrofitting work.

2.3.2. Traditional strenghtening of masonry walls

The actual practice of retrofitting is based on surface treatment, grout injection, external reinforcement, confining, center core method or post tensioning. In this section some of the most important features of the solutions will be described and discussed.

2.3.2.1. Surface treatment

It is one of the most common methods, widely applied through experience. It incorporates techniques as: ferrocement, reinforced plaster and shortcrete. These techniques are based on covering the masonry exterior and have a negative influence on the architectural appearance of the structure. From the structural point of view, this type of intervention assures an increase in strength, but also a complementary highly increase of the stiffness, thus sometime inducing negative behavioural aspects in the event of earthquake action.

Ferrocement

Ferrocement consists of a closely spaced, fine rod mesh (see Figure 2-87). The mesh is mainly made of metallic material, but can be of other low cost fibers. The mesh is fixed to the surface of the wall by mechanical anchors (6mm disposed at 250-400 mm) and then covered in high strength cement-mortar (10-50mm), achieving a reinforcement ratio of 3-8% [62]. This technique improves the tensile strength and realize a confining the masonry. In-plane and out-of-plane strength of the wall are increased. The experimental investigation carried out by Abrams &

Lynch shows that in plane strength can be increased by 1.5 times [62]. It is used for low cost houses and can be done with unskilled workers.

(a)

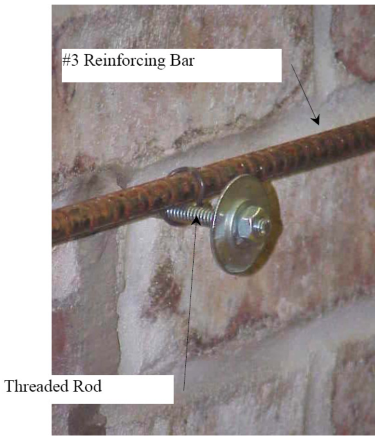
(b)

Figure 2-87. (a) Ferrocement mesh [62] and (b) example of rehabilitation [78]

Shotcrete

Shotcrete consist in an overlay of concrete sprayed on the surface of the masonry wall, over a mesh of reinforcing bars (see Figure 2-88). The thickness of the resulting cover can be adapted to the strength requirement, but usually exceed 60mm. The reinforcing mesh is usually made of welded bars designed to achieve effective crack control.

(a)



(b)

Figure 2-88. (a) Spraying shotcrete on masonry wall [62], (b) dowel to improve bonding [78]

Steel dowels are used to achieve an effective bonding between the existing masonry and the newly placed shotcrete layer [78]. Experiments by Khan [105] have shown that dowels don't improve the masonry – reinforced mortar composite panel's response. As a solution, many practitioners proposed to improve the bonding, by painting the masonry surface with epoxy before the shotcrete is applied

[62]. Also, by wetting of the masonry wall would improve bonding, but it has been reported that such procedure does not affect the cracking or the ultimate load [62].

By forming a parallel load bearing element on the surface of the existing wall a 3 to 25 times increase of the initial strength will be obtained [62] by using a 90 mm thick shotcrete. This technology builds "a reinforced concrete pier with no evidence of composite action with the masonry" [78] so that the contribution of the masonry wall can be completely neglected. In addition to a spectacular increase of strength, large deformation capacity and energy dissipation to the wall due to the yielding of the reinforcing bars are observed [62], [78]. There remains the disadvantage of a dramatic increase of the initial stiffness.

Reinforced plaster

Reinforced plaster is normal cement plaster, applied over a high-strength steel mesh [62]. By applying reinforced plaster, both the in-plane and out-of-plane strengths of the masonry can be increased. Depending on the quality and thickness of the steel mesh and of the plaster, there can be obtained an increase of strength from 1.25 to 3 times [62].

2.3.2.2. Grout injection

Grout injection has the advantage of not affecting the architectural appearance of the building. The method is used for re-establishing the bond in the cracks of the wall and to restore the original state of the wall, without an increase of the initial performances. Only in the case of double layer masonry walls, both brick and stone, the filling the internal cavity with cement based grout can increase the strength of the wall, by ensuring composite action between the layers of the wall [62]. Similarly, the rubble core of cavity walls can often be strengthened by injection [104]. Usually the rubble core is seriously destroyed by water infiltration, and there are enough cavities for the efficient consolidation of the walls.

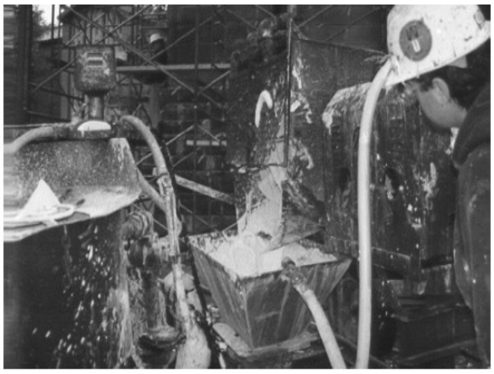
Some steps of the work phase of the grouting are presented in Figure 2-89.

The composition and consistency of the grout depends on the application. Epoxy resin is used for fine cracks of up to 2mm, while cement based grout is recommended for larger cracks and voids. It is important that the physical properties and the chemical composition of the grout should match the properties of the masonry [62].

Generally by cement grout injection the strength of the wall can be recovered up to 80-100% of the original (i.e. un-cracked) strength, or sometimes an increase of up to 40% can be achieved. If epoxy resin is used, an increase of strength (2-4 times) can also be achieved [62]. The change of stiffness is insignificant (10-20%) and should not affect the applicability of the method. In the case of filling the cavity walls with cement based grout, the strength gain can be as significant as 25-40 times [62].

(a) Flushing the core with water

(b) Pumping foam into mortar mixer



(c) Transferring grout to pump hopper

(d) Injecting grout into inner core rubble

Figure 2-89. Application phases for grout injection [104]

2.3.2.3. External reinforcement

Diagonal steel strips

Diagonal braces (steel strips, steel tubes of FRP laminates) can be pin connected on the face of the wall as external reinforcement of the URM (Figure 2-90) transforming the structure into some kind of X-braced frame, once the masonry is cracked and the system starts working. The external elements take over the tensile stresses through one brace and one vertical strip; compression is taken over by masonry at the opposite end of the wall [202]. This technique can improve the capacity up to 4.5 times [62] [202]. The out-of-plane strength of the wall is obviously improved. The failure mechanism of the wall subjected to shear is the crushing of the wall toe in the compressed part. At drift values of 1-1.5%, buckling of the vertical and the bracing compressed strips was observed [202] without affecting the strength of the system.

The masonry will undergo significant cracking before the steel bracing starts being effective [62], so, in order to obtain an efficient behaviour of the steel bracing system, a balanced ratio of rigidity between the virgin wall and the added bracing system must be obtained.

It is also important to note that the system need very strong, and stiff, fixing of the bracing ends (Figure 2-90a). In real applications, it is almost impossible to achieve connections of such strength, thus a lot of practical limitations arise.

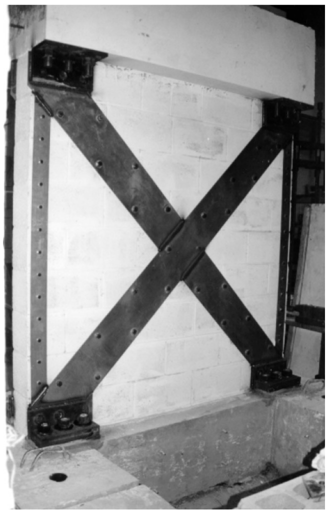


Figure 2-90. Wall strengthened with steel X-braces

Rectangular mesh of steel strips

A simple and efficient method based on mesh steel strips was experimentally tested (Figure 2-91.a) by Farooq *et al*. [69]. The experimental program included the reference virgin wall and walls strengthened on one side with a finer mesh or a coarser mesh and strengthened on both sides with the coarser mesh. Tests were carried out on the four configurations of walls, using a pre-applied vertical load of 18 tons.

(a) Wall strengthened with steel mesh (b) typical failure mode of the walls

Figure 2-91. Steel strips reinforcing [69]

The shear strength of the walls increased by 30-40% for the one-sided strengthening and by 87% for the two-sided strengthening. An increase of the masonry compression strength in the range of 12-26% was also reported [69], with the effects of delayed micro-cracking, and a consequent increase of the elastic limit of the walls.

The method can be easily applied, and it does not require special technology or qualification of the workers.

Three-dimensional tying systems

This method proposes to tie masonry parts by 3–D tying systems. Such a proposal based on stainless steel (CAM) is described in [57]. The stainless steel ribbons with a thickness of 0.8mm and width of 20mm are arranged as horizontal and vertical ties. The direction of the ties can vary and can be easily adapted to random arrangement of construction blocks encountered in old buildings proving versatility. The wall is tied as a "sandwich" from both sides and around the penetration holes at 125x125mm, and a 4 mm thickness plate is placed. Pre-stressing of the ribbons can also be applied, thus making possible to induce a tri-axial state of stress in the masonry.

(a) (b)

Figure 2-92. Application examples of CAM

In [57] a testing program is described on a small scale (90x90x12cm) wall specimens loaded in shear. Results show that, after initial cracking, they were able to restore the initial strength of the walls by CAM strengthening. Even a slight strength increase could be observed in the range of 15-50%. Because of the yielding of stainless steel, the assurance of a uniform cracking pattern for the wall and "corset effect" an increase of ductility and energy dissipation of 30-60 times more was obtained.

2.3.2.4. Confining masonry in RC tie columns and beams

The most used procedure to increase masonry buildings earthquake performance is the confining in reinforced concrete (RC) tie columns and beams. The method is prescribed by the design codes and used in the case of new buildings in most earthquake prone regions from South- America, Asia and Eastern Europe.

Vertical tie-columns are placed at every corner of the walls, at wall ends and door openings. These columns have the role to divide the wall surface, being disposed at intervals given by the design code (e.g. every 4m), and are tied horizontally by weak beams at every floor level or at regular intervals (e.g. every 3.5m). The columns and beams are essentially reinforced concrete columns and beams reinforced by 4 steel bars of Ø8-16mm (Figure 2-93). The disposal distance and reinforcing of the elements are mainly based on practical experience.

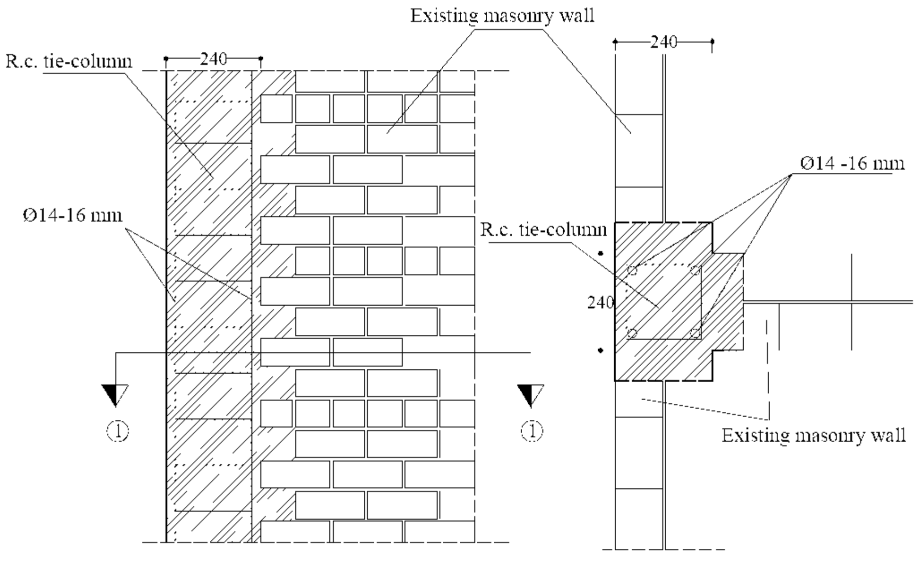


Figure 2-93. Example of tie-column arrangement in wall intersection [62]

The RC concrete provides a framing to the masonry, enhancing out-of-plane stability and confining the masonry for in-plane shear loading. The confinement effect shows its efficiency after masonry cracking, protecting the masonry from disintegrating. The bearing resistance of the wall is 1.2-1.5 times increased and most important, the ductility and energy dissipation are improved by 50% [62]. Again the performance of the system depends on the rigidity ratio between the masonry wall and the RC "frame".

This method is the main approach so as to pose new building to resist at earthquake action but the labour requirements are high in the case of retrofitting works by the removal of entire sections of the existing masonry in order to insert the RC elements, which causes long interruptions of the occupancy of the building.

2.3.2.5. Center core method

A reinforcing bar is mounted in the center of a vertical hole, of a 50-125 mm diameter, drilled in the middle of the wall thickness on the entire height down to the foundation (Figure 2-94.a). Afterwards, the hole is filled under pressure with cement based grout, polymer-sand or epoxy-sand mixture, for uniform filling of all voids. Mechanical fixings can be provided along the height at floor levels in order to anchor the floor and the roof to the newly created "strong"-column [62].

The procedure increases the lateral resistance of the wall to in-plane loads, mainly by providing a strong anchorage in the uplift regions and also slightly improves the out-of plane behaviour.

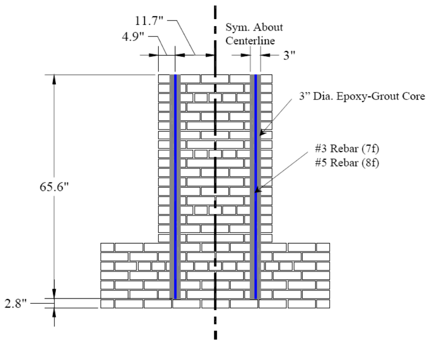
(a)

(b)

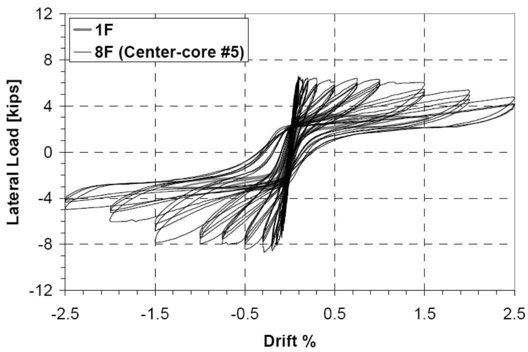
Figure 2-94. (a) Holes drilling and (b) grouting for centre-core rehabilitation in laboratory [78]

Experiments carried out on masonry piers subjected to medium level vertical loads and shear have shown that, by the centre core method, the strength of the masonry can be doubled [78].

Bending was the failure mode for both tested specimens and the energy dissipation was limited (see Figure 2-95.b). The ultimate lateral displacements achieved by the specimen (Figure 2-95.b) are encouraging, but this may be influenced by the geometry of the pier (Figure 2-95.a), not by the strengthening method used.



(a)



(b)

Figure 2-95. (a) Dimensions and (b) characteristic curve of centre-core reinforced masonry pier [78]

The main advantages of the centre core method are related to the untouched architectural aspect of the building and to the fact that the intervention can be executed externally. The main disadvantage arises from the fact that highly qualified personnel, high tech equipment and strict quality control are needed and from the structural point of view this technique creates zones with highly different stiffness and strength properties.

Together with confining, the introduction of hidden lamellar RC diaphragms inside the masonry wall thickness was the most used method in Romania in the case of old buildings, especially religious ones.

2.3.2.6. Post tensioning

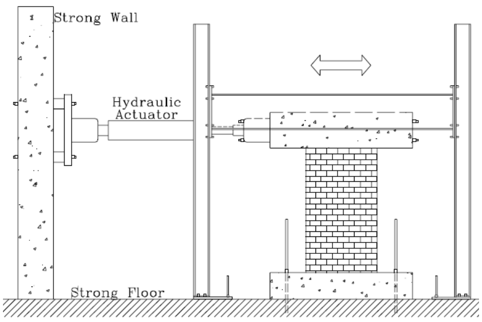
Internal post-tensioning

This technique can be found also in the case of old construction as a construction method, in which the masonry wall is built in the traditional way, and, after completion of the wall compression stresses are introduced to the masonry by the use of pre-mounted tendons in order to counter-balance the tensile forces.

Masonry poses a reasonably compressive strength, but very small tensile strength, so the idea of post tensioning comes naturally. The techniques used to achieve the post-tensioning are very numerous and based on tendons. Tendons are mostly steel cables or of carbon fiber [6]. Similarly to the center core method, tendons can be placed in holes, drilled into the masonry, grouted or not. In some cases [7], the masonry blocks are shaped in a way that the wall could be built around the tendon, without the need to introduce it in special holes (ducts) in the blocks.

The un-grouted tendons present the possibility of periodically verification of the tensile force level and the required modification may easily be done.

Cyclic tests on five configurations of post-tensioned walls with 3 tendons and an aspect ratio of 2/1 [189] emphasise the fact that strengthening of the compression part must be done for ensuring a good behaviour. In [7] this has been achieved by grouting the orifices of the concrete blocks and providing confining steel plates in the toe area of the wall. The beneficial effect of supplementary mild steel reinforcements is also underlined (damping increase from 7% to 10-12%). Unfortunately, even if, under laboratory condition and testing specimens such perfromance was obtained, is difficult to achieve the same condition in a rehabilitation work.



(a)


(b)

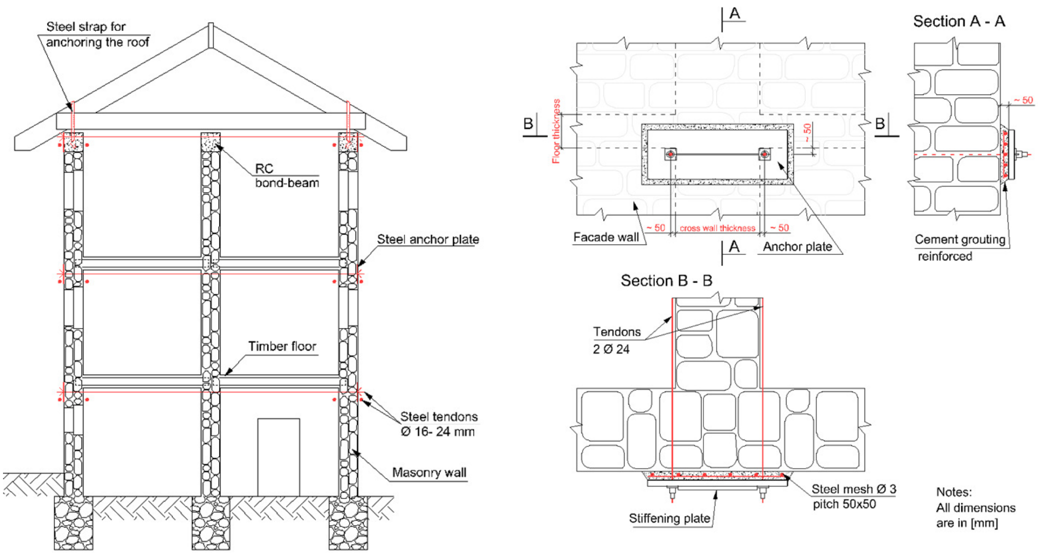
Figure 2-96. Test configuration of post-tensioned walls [189]

Many studies on post-tensioned masonry walls were carried out on concrete-block masonry (e.g. [223],[116]), but on a limited number on clay-brick masonry [189]. Anchorage in masonry is more complicated than in RC and the need for high technical requests and qualified workers lead to few real applications. Although some predictions made on basic calculations have shown that horizontal post-tensioning improves the resistance until now the experimental tests have not proved this assumption yet.

External post-tensioning (binding)

As previously discussed, an important prerequisite for achieving a good 3D structural behaviour is the binding of different components of the masonry structure. This requirement, with special focus on joints, is also discussed in chapter 2.3.1 of this document.

Opposite walls can be connected with steel or FRP rods (Figure 2-97). The rods may be post-tensioned for better effectiveness. Similar strengthening method is traditionally used against opening of arches due to outwards compression.



(a) (b)

Figure 2-97. Binding of opposite walls. Layout of the rods (a) and fixing details (b) [101]

The effect of post tensioning rods greatly depends on the configuration of the building and on the way rods are disposed. Few systematic studies of this method exist; some are unusual and are proposed only for low-cost structures [209].

Beside technical difficulties, the biggest disadvantages of the method are that external straps and connections might affect the architectural aspect of the buildings, and the post-tensioning elements, being external, are exposed to corrosion.

2.3.3. Strengthening of masonry walls based on FRP

In the past decade fiber reinforced polymers have know a continuous growing and have started to become one of the most used and attractive retrofitting technique in the case of masonry walls, thus improving the behaviour of most critical failure modes. The high resistance, the light weight and the rather simple application technologies recommend FRP to be used in most cases.

On the market there are a wide range of typologies. The most used are Glass, Carbon and Aramid Fiber Reinforced Polymers (GFRP, CFRP and AFRP), but also PolyVinylAlcohol Reinforced Polymers (PVAFRP) [217] have been used. The binder material (i.e. matrix) in which reinforcing fibres are embedded is the epoxy resin.

The application of FRP knew many forms: Uni-Directional (UD) laminate strips, bidirectional fabrics and Near Surface Mounted FRP bars, rods or strips [25].

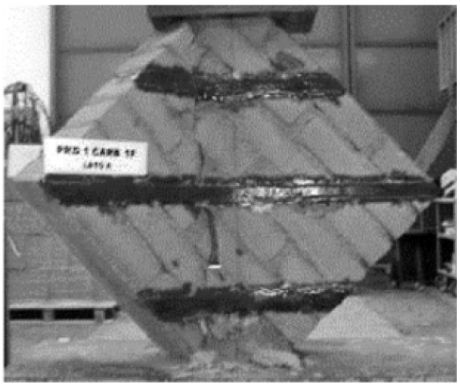
Many studies, that have been carried out, have reported an important improvement of masonry wall behaviour for in-plane [206], [217], [63], [164], [180], [86], [218], [28] and out-of-plane [206], [208], [24], [164], [92], [112], [218], [25] behaviour. This thesis will make only a short overview of FRP application possibility without detailing.

2.3.3.1. Uni-directional FRP strips

Unidirectional (UD) FRP in X assemblies

UD FRP strips have been utilized in various assemblies. Perhaps the most typical assembly for shear strengthening is the X assembly (Figure 2-98, [63]) with two strips or wider plates along the diagonal of the wall [77], [63], [164], [28].

Various experimental studies have reported an increase of 15-65%, on one side, and 45-75%, on both sides, in terms of of strength and from 40 to 240% increase in terms of displacement.

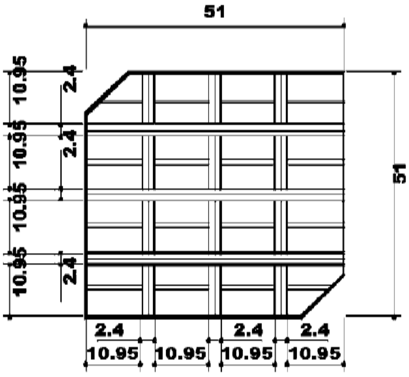

(a)


(b)

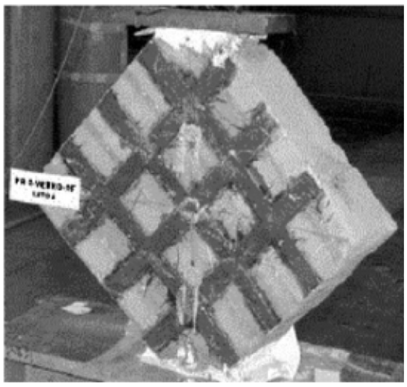
Figure 2-98. (a) Testing of wallettes [217]; (b) FRP assembly with UD laminate placed in X [63]

Rectangular unidirectional (UD) FRP grids

Rectangular grids (Figure 2-99.a [217]), vertical strips or plates [63], [164], [180], especially near both ends of the wall or at the jambs of a pier, horizontal strips [164] and also several parallel diagonal strips (Figure 2-99.b [217]) have also been used for retrofitting URM.



(a)



(b)

Figure 2-99. Rectangular FRP grids :(a) typical grid configuration and (b) test results by [27]; 2 vertical UD CFRP plates placed at ends on one face [28]

Compared to the X assembly, the rectangular grid offers better stress distribution that causes crack spreading and a less brittle failure. In most cases less stiff FRP material appeared to be more effective both in terms of ultimate strength and stiffness increase.

Rectangular UD FRP strips are also very effective for out-of plane (i.e. bending) strengthening of URM. Galati *et al.* [81] reported that, if the wall behaves as a simply supported element (out-of plane) external FRP reinforcement is very effective, but for fixed end condition the effect is limited, due to stronger "arching effect"

2.3.3.2. Bidirectional FRP laminates

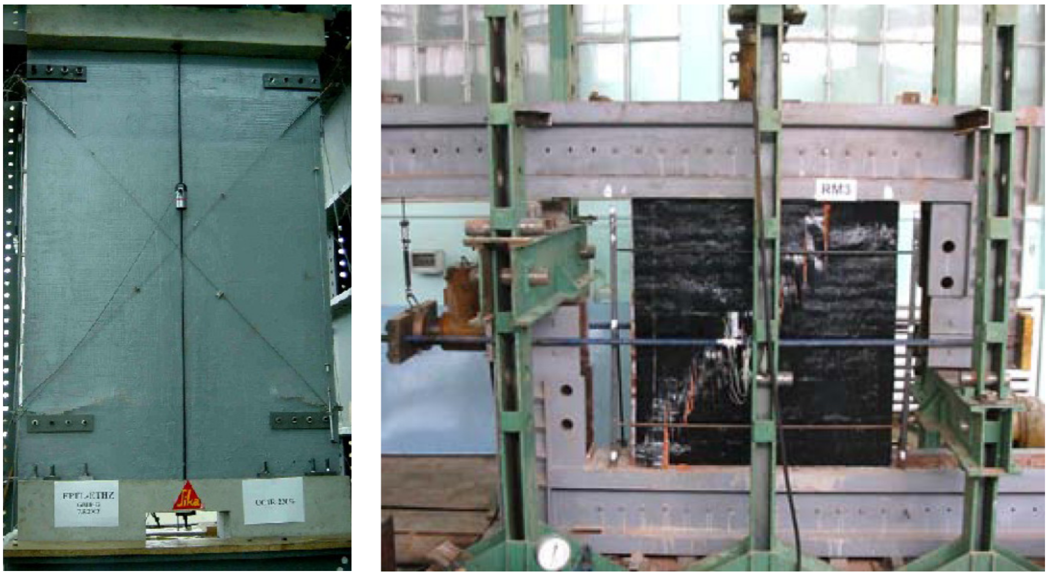
Bidirectional laminates [63], [180], [218] can been used either to cover the entire face of the wall or only limited to the piers.

Depending on the geometry of the wall and the expected failure mode, various experimental studies on FRP bidirectional (alone or with complementary measures) have reported an increase of 15-470% on one side and 230-1000% on both sides, in terms of strength and up to 1000% increase in displacement.

Good surface preparation and careful lamination was found to be very important. If an initial imperfection or separation occurs in the fiber overlay due to an entrapped air bubble or fault during fabrication, premature failure of the fiber overlays may occur due to stress concentration resulting from rapid propagation in the joint opening adjacent to the faults.

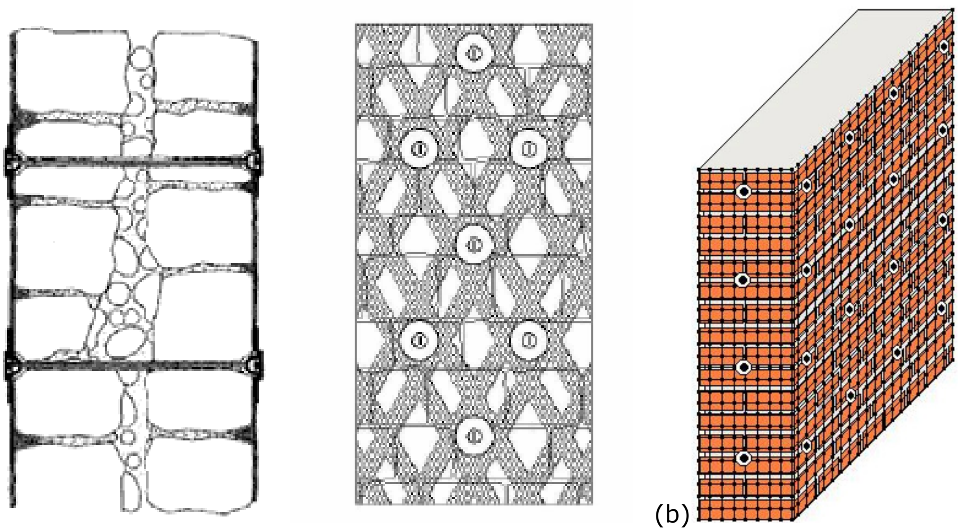
2.3.3.3. Polymer grids

The external reinforced laminates can be preimpregnated with epoxy resin and connected to the masonry elements with or without confinement connectors (Comrehab Figure 2-101a and RichterGard systems Figure 2-101b). It can resist high tension, and although it is slender its compression resistance can be improved by gluing it to the wall and prestressing it with the connectors and tie rods through the wall.



(a) (b)

Figure 2-100. FRP bidirectional laminates. (a) rocking failure with rupture of FRP [28]; shear failure of wall strengthened with FRP fabric with vertical fibers [166]



(a) (b)

Figure 2-101. Polymeric grids (a) COMREHAB (b) RichterGard

The reinforcement laminates are not ductile by themselves. If there is connection to the wall by tie rods, the ductile behaviour of a new composite system is improved due to the confining effect.

2.3.3.4. Near Surface Mounted techniques

Near Surface Mounting (NSM) is a new technique in which FRP bars or FRP UD strips are installed into slots grooved into the wall surface (masonry or concrete) and the slots are then filled with epoxy based or cementitious grout. In case of masonry, FRP are installed in slots grooved in mortar joints and is called FRP structural repointing.

NSM FRP can be used as horizontal [206], [112] and vertical [206], [24], [112] reinforcement. The advantage is related to the facts that it requires no surface cleaning or levelling and the change in the appearance of the structure is minimal.

(a) (b)

Figure 2-102. (a) Installation of NSM GFRP bars; (b) test set-up of wall [207]

In-plane behaviour is improved in terms of resistance, but mostly in terms of deformation capacity and energy dissipation, allowing a ductile failure mode.

The efficiency of the NSM FRP in out-of-plane bending strongly depends on the configuration of the masonry wall and the anchoring strength, fact underlined by the wide scattering of experimental results.

2.3.3.5. Strengthening of toe by FRP confinement

Sometimes FRP has also been used at the toes (lower corners of the wall of the first floor) of slender walls to protect the toes against crushing. The FRP fabrics used for strengthening have mostly been stitched fabrics, but woven fabrics have also been used [77] [92].

2.3.3.6. Strengthening of mortar joints

In old masonry the mortar is often very weak, and several authors have suggested ways to improve the strength of the mortar joints by using of polymer fibers. Zhu and Chung [225] studied the improvement of brick-to-mortar bond strength by the addition of short carbon fibers to the mortar. 110% increase in bond strength under shear loading and 150% improvement in bond strength under tensile loading were obtained. Sofronie [197] presented a method for strengthening the mortar joints by polymer grids (RichterGard see Figure 2-103) and stated that this method strongly improves the ductility of the mortar joints. Anagnostopoulos & Anagnostopoulos [15] studied polymer-cement mortars for the improvement of the mechanical properties of ancient masonries. Large improvements in flexural and shear strength were obtained by mortars with high latex content. Burdette *et al.* [32] and Straka [199] studied FRP ties for masonry walls. FRP ties were found feasible, but fire safety was stated to be a problem. The smaller energy dissipation compared to steel ties was not discussed in these papers. The creeping of polymers may also become a problem.

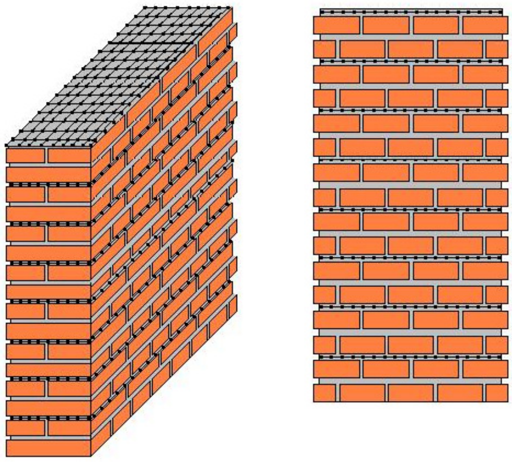


Figure 2-103. Mortar joints strengthened with polymer grids

Textile reinforced mortars (TRM)

A special method is the Textile Reinforced Mortar (TRM). In TRM the epoxy matrix is replaced by cement based mortar [179] in external strengthening of walls. This alternative may be feasible in the case of high temperature exposure or the strengthening is to be done on wet surfaces or under low temperatures.

Papanicolau *et al.* [179] compared TRM with FRP as strengthening material for URM in static cyclic out-of-plane loading. The failure occurs in the textile (tensile failure) when FRP is more effective than TRM; but if the failure occurs in the masonry (compression/shear failure), the TRM is more effective.

Polymer grids

The strengthening of mortar joints with polymer grids increases the tensile strength and ductility of the masonry [197]. This method can be used for the retrofitting of existing walls only if the existing mortar is partially or totally removed. This method is applicable when the strengthening wall-to-wall, wall-to-column or wall-to-foundation joints is required.

2.3.3.7. FRP – advantages and disadvantages

In the last decade, FRP materials have been widely treated, tested and applied in real application cases. These materials pose important advantages as: very versatile, high-strength, lightweight, non-corrosive and relatively easy to apply on URM. Some disadvantages must also be noted: they are expensive compared to steel, non-ductile, have smaller modulus of elasticity, at cracking strain of the masonry the stress level in the FRP is very low, compared to the strength of the material, sensitivity to temperature (some epoxy resins start softening at 45-70ºC) or local failure by screeching and very often their anchoring creates serious problems. In some cases, the advantages of easily applying FRP are lost, due to the heavy interventions needed for the anchoring.

There can be found competitive steel solutions equivalent to the strengthening techniques using FRP. One such case is NSM FRPs. Steel ropes or cold-formed strips could be used as NSM, with advantages over FRP: an increased cracking strength of the masonry having a large elastic modulus, a larger deformation capacity in the cracked state, possibility to increase the adherence to the masonry by using profiled steel strips, and a significantly lower cost.

The major weaknesses of URM are the complete lack of tensile strength of the material and lack of deformability. Therefore, the intervention usually consists in providing this missing tensile strength. Based on data from [33], Figure 2-104 shows the "price" of providing tensile strength with different materials. As it can be observed, steel is very competitive both in the low (Group I) and high (Group II) tensile requirement regions. FRPs compensate with lower installation costs and intervention times (i.e. by reducing interruptions in the use of the building).

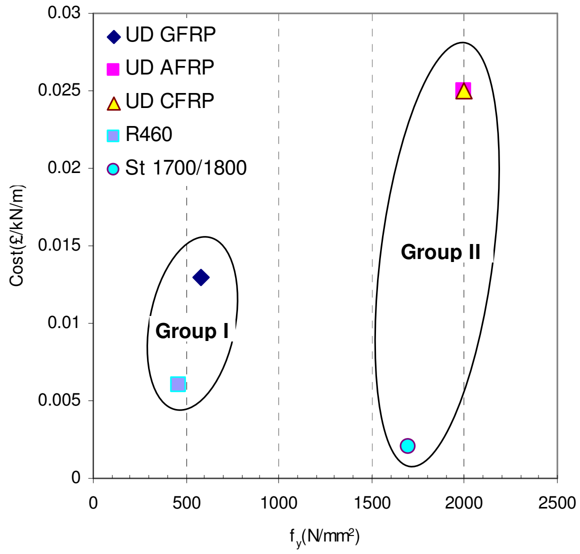


Figure 2-104. Costs of providing tensile strength in UMR strengthening work [80]

Most probably, these two costs are more significant in the rehabilitation works, and less important in the strengthening works. Therefore, FRPs are probably more competitive in the rehabilitation interventions, and steel can be more competitive in strengthening.

From the point of view of deformability and energy dissipation, steel has an obvious advantage as a material, but in the case of retrofitted masonry elements, the anchorage measure clearly plays an essential role. This advantage is not entirely used, because although steel can support more displacement the masonry element cannot support anymore.

2.3.4. Advanced techniques

Venice Chart article 10: "Where traditional techniques prove to be inadequate, the consolidation of a monument can be achieved by the use of any Modern Techniques for the conservation and construction, the efficiency of which have been shown by scientific data and proved by experience".

Masonry walls usually have very low in plane deformability and are characterized by lack of deformability. Only very slender walls, with a failure mode dominated by rocking can develop more significant horizontal deformations without sudden and dramatic decrease of the load bearing capacity. Therefore, any passive structural control system attached parallel to the wall would not be efficient.

It is difficult to accommodate any passive control devices to masonry buildings. If passive system form a separate load bearing system or is apply to very slender walls can be effective. Also placing at the foundation/roof/floor and walls interaction level can be effective. Base isolation technique is for sure a very effective seismic protection tool in case of stiff masonry structure. Cost and hard complementary labour as new foundations limit there application in case of existing buildings.

Generally passive control devices allow cracking before activation of system and offer significant deformation capacity after cracking. An amount of damage to the structure must be accepted and the devices should by used only as "collapse-prevention" tools in the damage control range.

Also regarding to active control, it is doubtful how this type of active device will function in case of masonry structure.

2.4. COMMENTS AND CONCLUSIONS

Nowadays, two approaching methods are known for seismic rehabilitations: one is based on enforcing the elements or the overall structure and the other one tries to control the seismic response. According to the first approach, each structural element is properly enforced so as to correspond to an increased degree of safety. The methods to accomplish this are generally based on traditional techniques and they generally try to use materials identical to the ones present in the existing building. This approach was used on a large scale for repairing as well as for antiseismic conformation of structures and it is still in use today. Even if these methods offer, in many cases, a good seismic protection level, they have some flaws regarding technical and economical aspects:

The traditional methods of enforcing based on materials affinity improve the materials behaviour but does not change by much the structure's seismic response. Moreover, strengthening or increase in rigidity of some elements can lead to a unfavourable behaviour at seismic action, especially when the intervention is limited to a number of elements.

The traditional systems of strengthening generally require a high amount of materials in order to obtain a satisfactory seismic performance. For example, the reinforced concrete structures require an increase in cross-sectional area. This fact leads to increased inertial masses, as well as to the compromising of the building functionality.

Lately, new products have been introduced on the market with the purpose to repair the deteriorated concrete or masonry elements and, in the same time, to increase the mechanical behaviour in terms of ductility and resistance. The new products are part of the epoxy family, resins and reinforced plastic fibres (GFPR – glass fibre reinforced plastic) or carbon (CFRP – carbon fibre reinforced plastic), together with other synthetic fibre based products used in order to confine concrete or masonry elements. Recently, applications of a more advanced and sophisticated systems have been proposed in order to reduce the seismic vulnerability of buildings, including active or passive control of seismic motion induced vibrations. BIS (base isolation system) is one of the most efficient alternatives regarding the seismic response of the structure. The method consists in inserting deformable elements at the structure's base. Thus, the dynamic characteristics are substantially modified. This method implies the total mechanical decoupling of the superstructure and foundations in order to insert the isolation devices. The most used are the elastomeric devices, which act as mechanical filters, considerable reducing the magnitude of inertia forces acting on the superstructure.

Although they have all these advantages, these kinds of techniques still have some flaws, like:

The innovative systems based on FRP were not sufficiently studied in the domain of consolidations and especially regarding the long term efficiency and behaviour. This fact does not refer only to their durability, but also to their compatibility with the existing materials, in particular, in the case of monumental buildings.

In the case of base isolation systems, although they are very efficient when it comes to improving the seismic response, the demand of perimetral "cutting" of the building leads to the necessity of a new foundation system in order to achieve the connection between all superstructure elements. This fact leads to supplementary work, as well as to complications in terms of technological procedures of setting up. Moreover base isolation systems are not suitable for tall buildings like towers. The complexity and the high prices of BIS make this system more suitable for new buildings, when a higher degree of protection is required, than for the rehabilitation of existing buildings.

Moreover all these techniques are irreversible, which means that once applied to an existing building they cannot be removed. This is the main why their use is not suitable for monumental buildings.

The situation presented before tries to bring into the light the current situation in the field of rehabilitations and to underline the difficulty raised by this kind of interventions.

The structural methods of intervention must take advantage of all technological systems and modern materials, in such a way that it could offer not only a statical solution for the problem but an optimal economical solution, functional and aesthetically compatible with the existing building, technically achievable considering the transport requirements, the setting up and the activity in narrow spaces.

Some comments regarding the strengthening of masonry walls are useful in order to identify the features that should pose a new and efficient technique. The comparison of different strengthening methods for unreinforced masonry (URM) is not an easy task because very small changing in masonry typology, load application, vertical precompresion, boundary condition and experimental set-up can greatly change the results.

It is believed that in the case of testing retrofitted walls in-plane loads application should be static monotonic and cyclic with larger steps in amplitude between cycles [80].

The in-plane strengthening of an URM wall can be deliberately aimed at changing the failure mode from diagonal shear failure to in-plane flexural failure (rocking) that involves alternating uplift and crushing of the corner (toe). Rocking supplies large horizontal deformation capacity and, provided that crushing of the toe occurs in a relatively small area, it does not lead to sudden loss of capacity like diagonal shear failure. It is also a self-centering failure mode.

Any retrofitting techniques of masonry walls must take into account that if they only increase shear strength without adding to the deformation capacity and to energy dissipation, they may be disadvantageous. When maximum force is be reached, sudden brittle failure can occur causing fast load redistribution to adjacent elements and may trigger collapse or the falling of loose material released in the sudden failure, causing injuries or death.

Sometimes, the attempt to suppress the shear failure using a retrofitting system (e.g. vertical FRP strips, vertical post tensioning with steel rods) that causes large tensile stresses may reduce the capacity of the wall against rocking or sliding. Some retrofits (e.g. horizontal FRP strips) are aimed at forcing rocking or sliding instead of shear failure, because these failure modes occur at higher loads than shear failure and provide significant displacement capacity. Anyway, although good experimental results for single wall on rocking or sliding are obtained, in 3D assembly this will cause large bending or torque loads to the adjacent walls and together with the vertical cracks that will appear near the corners may lead to out-of-plane bending failure of the adjacent walls. The out-of-plaine failures are sudden and cause injuries or death.

A limitation in the existing research is that it has been too much strength oriented, by sometimes neglecting the ductility and energy dissipation and the studies were limited at one wall without showing any particular attention to adjacent elements that will not be able to accommodate at impressive gain in strength or displacement of in-plane tested elements. The larger load attracted due to the stiffness increase of the strengthened wall has often been neglected.

A comparison of the different methods is presented in the following tables. The criteria used are:

The availability of the rehabilitation method for different configurations of masonry, and the major limitations in the use of each method (Table 2-4);

The main failure mechanism affected/improved by the rehabilitation method, representing the “goal” of the intervention. Very often secondary/unintended mechanisms are also acting. They are also mentioned. (Table 2-5);

The possible performance improvements, in terms of strength gain, stiffness gain and energy dissipation increase, reported in the reviewed literature. In the same Table 2-6, a comparison of the economic impact of each intervention is represented. The economic aspects of the different methods can only be quantified in specific case studies. It is also probable that different methods will be more economical in different case. Therefore, Table 2-6, only gives a qualitative assessment of the economic impact.

Table 2-4. Suitability for wall typologies and main limitations of rehabilitation method [80]

	Single leaf walls	Cavity walls with rubble filled core	Bonded brick-work	Stone masonry walls	Light-weight CMU units	Concrete block walls	Brick column	Stone column	Joints	Applicability on irregular or rough surfaces	Applicability with weak adjacent material	Visibility for workmanship quality control	Chemical and environmental durability	Fire safety	Aesthetic change
Ferro-cement	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	A	A	G	IM	G	S
Shotcrete	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	A	SC	G	IM	G	S
Reinforced plaster	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	-	NA	SC	G	IM	IM	S
Grout injection	Yes	Yes	Yes	-	Yes	Yes	Yes	-	-	A	A	P	IM	G	-
Diagonal steel strips	Yes	Yes	Yes	-	-	Yes	-	-	Yes	SC	NA	G	IM	P	M*
Rectangular mesh of steel strips	Yes	Yes	Yes	-	-	Yes	Yes	Yes	Yes	NA	SC	G	IM	P	M*
3D steel tying	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	A	SC	G	P	P	M
RC tie columns and beams	Yes	Yes	Yes	Yes	Yes	Yes	-	-	Yes	A	SC	G	G	G	S
Centre core reinforcement	Yes	-	Yes	Yes	Yes	Yes	Yes	-	-	A	A	P	G	G	-
Internal post-tensioning	Yes	-	-	Yes	SC	Yes	Yes	-	-	A	A	P	G	IM	-
External post-tensioning	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	A	SC	G	P	P	M
UD FRP in X	Yes	Yes	Yes	-	Yes	Yes	-	-	Yes	NA	NA	G	G	P	M*
UD FRP rectangular grids	Yes	Yes	Yes	-	Yes	Yes	-	-	Yes	NA	SC	P	G	P	M*
BiDir FRP laminate	Yes	Yes	Yes	-	Yes	Yes	Yes	-	Yes	NA	A	P	G	P	M*
NSM FRP	Yes	Yes	Yes	-	Yes	Yes	Yes	-	Yes	A	SC	IM	G	IM	S
Toe confinement	Yes	Yes	Yes	-	Yes	Yes	-	-	-	SC	SC	P	G	IM	-
TRM	Yes	Yes	Yes	-	Yes	Yes	Yes	-	-	A	A	IM	IM	G	-
Polymer grid	Yes	Yes	-	-	Yes	Yes	Yes	-	Yes	A	A	P	G	IM	-

NOTES: Yes Possible to use the method for both restoration and strengthening.
Int Only on the interior surface of the wall
* If the wall had plastering which can be remade then S or "-"

A – Applicable; NA – Not Applicable; SC – Special Care
G – Good; IM – Intermediate; P – Poor
M – Major; S – Small; - – None

Table 2-5. Failure mechanism improved by the rehabilitation method [80]

	Sliding shear	Oblique shear		Bending			Overall confining of the masonry	Connect layers in multi-wythe wall	3D interaction
		In mortar	In stones	Corner uplift, Anchoring	Tension cracking	Toe crushing			
Ferro-cement	Yes	Prim	Prim	If mesh is anchored	Yes	Yes	Yes	-	Yes
Shotcrete	Yes	Prim	Prim	-	Yes	Yes	Yes	-	Yes
Reinforced plaster	Yes	Prim	Prim	If mesh is anchored	Yes	Yes	Yes	-	Yes
Grout injection	Prim	Prim	-	-	-	Yes	-	Yes	-
Diagonal steel strips	Yes	Prim	Prim	If anchored	-	-	-	-	-
Rectangular mesh of steel strips	Yes if anchored	Yes	Yes	Yes if anchored	Yes	-	Yes	-	-
3D steel tying	Yes	Yes	Yes	Yes	Yes	-	Yes	Prim	Prim
RC tie columns and beams	Yes	Yes	Yes	Prim	Yes	Prim	Prim	Yes	Yes
Centre core reinforcement	-	Yes	Yes	Prim	Prim	Yes	-	-	-
External post-tensioning	-	Yes	Yes	Prim	Prim	-	-	-	Prim
Internal post-tensioning	Yes	Yes	Yes	Prim	Prim	-	-	-	-
UD FRP in X	Yes if anchored	Prim	Prim	Yes if anchored	-	-	-	-	-
UD FRP rectangular grids	Yes if anchored	Prim	Prim	Yes if anchored	Yes	-	Yes	-	-
BiDir FRP laminate	Yes if anchored	Prim	Prim	Yes if anchored	Yes	-	Yes	-	If connected
NSM FRP horizontal	-	Prim	Prim	-	Yes	-	-	-	-
NSM FRP vertical	Yes if anchored	Yes	Yes	Prim	Prim	-	-	-	-
Toe confinement	-	-	-	-	-	Prim	-	-	-
TRM	Yes if anchored	Prim	Prim	-	Yes	Yes	Yes	-	-
Polymer grid	-	Yes	Yes	-	-	Yes	Yes	-	Yes

NOTES: Prim – Primary goal of the intervention/failure mode mainly affected
Yes – Strength in this failure mode also improved, even if it was not primary goal of the intervention

Table 2-6. Possible performance benefits and economic consequences of the use of different rehabilitation techniques [80]

	Strength increase (%)	Stiffness increase (%)	Ductility increase (%)	Energy dissipation increase (factor)	Space use	Tearing down and rebuilding parts	Effect on existing finishing	Material cost	Work amount
Ferro-cement	50	I	I	I	IM	S**	L	IM	L
Shotcrete	200-500	I	-	-	L	S	L	IM	IM
Reinforced plaster	25-200	-	I	I	-	S**	L	S	L
Grout injection	0-40	10-20	-	-	-	S	-	IM	L
Diagonal steel strips	350-900	-	I	I	-	S**	IM	S	IM
Rectangular mesh of steel strips	40-90	-	I	-	-	S**	L	IM	L
3D steel tying	15-50	-	I	30-60	-	-	IM	S	IM
RC tie columns and beams	20-50	I	50	50	L*	L	IM	L	L
Centre core reinforcement	100	I	I	-	-	S	-	L	L
Internal post-tensioning	I	I	I	-	-	S	S	S	L
External post-tensioning	I	-	I	-	S	-	IM	S	S
UD FRP in X	50-200/800 (if FRP anchored)	I	50-100	I	-	S**	IM*	S	S
UD FRP rectangular grids	10-50	I	80	170	-	S**	L	S	IM
BiDir FRP laminate	100-1000	I	20-1000	I	-	S**	L	IM	IM
NSM FRP	10-80	-	45	35	-	-	S	S	S
Toe confinement	-	-	-	-	-	-	S	S	S
TRM	-	-	-	-	IM	S	L	IM	IM
Polymer grid	-	-	-	-	-	L	-	S	L

NOTES: I – Increased but values not reported in literature
L – Large; IM – intermediate; S – small - – not relevant
* If used externally. The value is "-" if the r.c. elements are placed within the wall.
** Replacement of the plaster.

3. PERFORMANCE BASED SEISMIC ASSESSMENT

3.1. INTRODUCTION

The need to ensure a satisfactory behaviour under seismic action for old buildings designed before the appearance of modern engineering rules or according to poor seismic provisions has become an important task that comes to the civil engineering community. In the light of recent knowledge regarding seismic motion and structural behaviour many of the existing buildings are obviously substandard and deficient. Older hazardous buildings are responsible for the thousands of life losses and significant damage. It is possible that the existing substandard buildings might actually outnumber the safe buildings. Therefore the attention in earthquake engineering should focus more on the existing buildings than on the new ones, in order to provide advanced methodologies for building assessment. In the afterlight of disastrous events the engineering community silently consented to upgrade existing buildings so that to reach the safety level of new buildings, according to the limit state procedure provided by standards. The implications of this concept were not at all rational, first from the technical point of view and second from the one of the cost and length of time. Thus a new approach has arisen based on multi-level evaluation together with differentiate targets [75].

Modern Design Recommendations, like the SEAOC's Vision 2000 project and the BSSC's NEHRP Guidelines for Seismic Rehabilitation of Buildings helped develop a new concept in building evaluation and design by introducing design performance objectives, acceptance criteria tied to the performance level, and the use of alternative analytical techniques for performance evaluation. The proposed 1997 NEHRP Provisions for Seismic Regulation of Buildings and Other Structures also made an important contribution, by attempting for the first time to **define the margin of safety inherent in buildings** conforming to these provisions, and in the sense of Ultimate Limits States design philosophy, by directly incorporating this presumed margin in the definition of the loading function [90].

Key areas of development, required in oreder **to provide true performance-based capability** in future design and evaluation provisions include the incorporation of a specific serviceability level performance procedure, verification of the reliability actually inherent in buildings of different structural systems conforming to the provisions and the development and refinement of new analytical evaluation (acceptability) procedures capable of predicting building performance with reduced uncertainty [90].

Retrofitting of all the vulnerable buildings before the next big earthquake is also not a realistic solution either. The highest priority should be on identifying the buildings which have a high possibility of collapsing and on identifying those which can ensure life safety despite being substandard. Seismic rehabilitation of large stocks of buildings requires engineering approaches different from the traditional approaches of civil engineering. Methodologies to evaluate the seismic risk of highly urbanized area are needed. In latest years important steps have been made from the development of quick methods to establish buildings vulnerability and assess seismic fragility [75].

Figure 3-1 presents the relation between Evaluation, Design and Construction within the *Conceptual framework* of Performance Based Seismic Retrofit (PBSR). It is important to underline the iterative design process in order to achieve or respect the desire performance level. In every phase of the rehabilitation process, the acceptability criteria play the decisional role, from the initial evaluation to the design process and the quality insurance for the erection process.

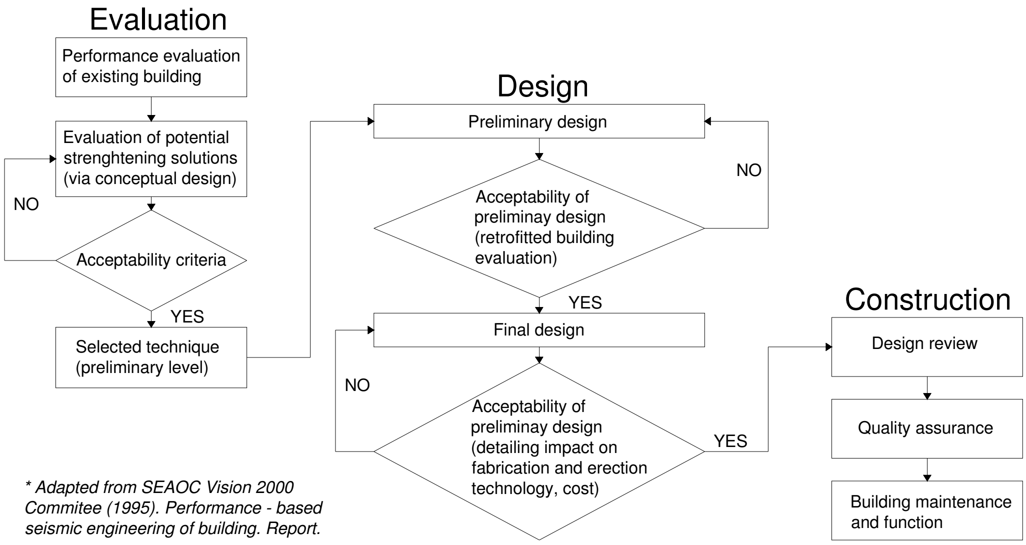


Figure 3-1 Conceptual framework for Performance Based Seismic Evaluation

3.2. PERFORMANCE BASED EVALUATION AND RETROFIT OBJECTIVES

The intention of Performance-Based Earthquake Engineering (PBEE) is to provide methods for designing, constructing, evaluating and maintaining buildings, so that they should be capable of providing predictable performance when affected by earthquakes. Performance is measured in terms of the amount of damage sustained by a building, when affected by an earthquake-type ground motion [90].

Inherently, the performance-based design concept implies the definition of multiple target performance (damage) levels which are expected to be achieved, or at least not exceeded, when the structure is subjected to earthquake ground motion of specified intensity. Much of the early development effort has taken place in the preparation of the NEHRP Guidelines for Seismic Rehabilitation of Buildings (ATC, 1996), intended as a resource document for use in upgrading the performance of existing buildings [90].

Though the name performance-based engineering is new, the basic concept of developing buildings and structures that will meet expected performance levels under different ground motion scenarios is certainly not. Design codes from all over the world indicated, more or less, that structures designed in accordance to their provisions would be able to meet some the of specific performance objectives, i.e. to resist:

- Moderate earthquakes with limited structural and non-structural damage;

- Major earthquakes with significant damage to structural and non-structural elements, but with limited risk to life safety.

These same basic performance objectives, though more precisely and quantitatively defined, are being adopted by most performance-based engineering guidelines today. In traditional practice, earthquake design has been explicitly performed for only a single design event level, at which a level of performance generally termed "life safety" has been targeted without providing any other specific procedures to allow the evaluation of the ability of a structure to actually meet other objectives. Contemporary efforts of performance-based engineering are seeking to provide reliable methods of meeting multiple performance goals through explicit design procedures [90].

3.2.1. Phases of Performance Based Evaluation and Retrofit

The Performance Based Seismic Assessment (PBSA) or Evaluation (PBSE) involves the following steps (Table 3-1) from strategy, to the concept and detail of the building retrofit work [19].

Table 3-1 Phases in PBSA process [19]

	Initiate the process	Jurisdictional requirements Architectural changes Voluntary upgrade
	Select qualified professionals	Structural Engineer Architect
	Establish performance objectives	Structural Stability, Limited Safety, Life Safety, Damage Control, Immediate Occupancy
STRATEGY	Review building conditions	Review Drawings Visual Inspection Preliminary Calculation
	Formulate a strategy	Simplified Procedures Inelastic Capacity Methods Complex Analyses
	Begin the approval process	Building Official Review
CONCEPT	Conduct detailed Investigations	Site Analysis Material Properties Construction Details
	Characterize seismic capacity	Modeling Rules Force and Displacement
	Determine seismic demand	Seismic Hazard Interdependence with Capacity Target Displacement
DETAIL	Verify performance	Global Response Limits Component Acceptability Conceptual Approval
	Prepare construction documents	Similarity to New Construction Plan Check Form of Construction Contract
	Monitor construction quality	Submittals, Tests and Inspection Verification of Existing Conditions Construction Observation by Designer

A PSBA procedure supposes the collaboration of all the involved parts with a specific implication in different phases of the process (see Figure 3-2).

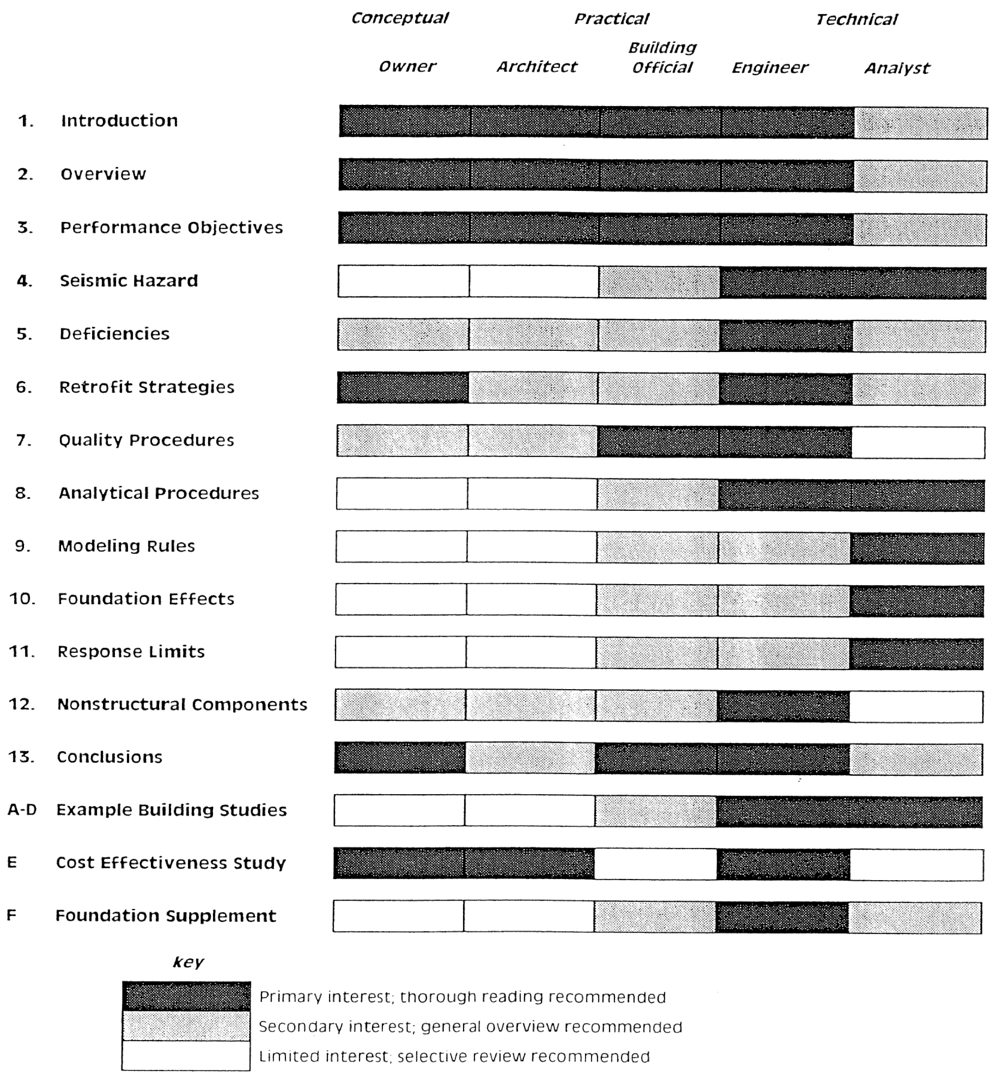


Figure 3-2 Audience interest spectrum (ATC-40 [19])

3.2.2. Building performance objectives

A *Rehabilitation Objective* consists of one or more rehabilitation goals, each goal consisting in the selection of a target Building Performance Level and an Earthquake Hazard Level [73] (see Figure 3-3). Thus the association of a performance level (damage state) to a hazard level is called a performance-objective [21]. Rehabilitation Objectives should be selected based on the building's occupancy, the importance of the functions occurring within the building, economic considerations including costs related to building damage repair and business interruption, and consideration of the potential importance of the building as a historical or cultural resource.

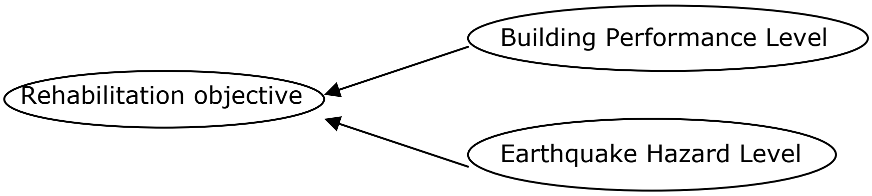


Figure 3-3 Establishing Rehabilitation Objective Principles

The building owner, in consultation with the designer, shall select a seismic Rehabilitation Objective but never bellow the code official provision. The selection of a Rehabilitation Objective will consist in of the selection of a target Building Performance Level, which intend to represent goals of structural behaviours, from a range of performance levels and in the selection of an anticipated Earthquake Hazard Level from a range of seismic hazards [73].

Difficulties in establishing performance could be associated with unknown geometry and member sizes in existing buildings, deterioration of materials, incomplete site data, variation of ground motion that may occur within a small area, and incomplete knowledge and simplifications related to modeling and analysis.

Building performance should be described qualitatively in terms of the safety afforded by building occupants during and after the earthquake; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs and economic, architectural or historic impacts on the larger community.

Different national's codes establish various Rehabilitation Objectives. These Standards more or less tackle with the same issues and establish Objectives that can be summarized according to **FEMA 356** [73] as:

- Basic Safety Objective (BSO)
- Enhanced Rehabilitation Objectives (BSE-1, BSE-2)
- Limited Rehabilitation Objectives
- *Reduced Rehabilitation Objective*
- *Partial Rehabilitation Objective*

Basic Safety Objective (BSO) is intended to approximate the earthquake risk to life safety traditionally considered. Buildings meeting the BSO are expected to experience little damage from relatively frequent, moderate earthquakes, but significantly more damage and potential economic loss from the most severe and infrequent earthquakes that could affect them [73].

Enhanced Rehabilitation Objectives (BSE-1, BSE-2) can be obtained by designing for higher target Building Performance Levels (method 1), at important building and facilities, or by designing with the use of higher Earthquake Hazard Levels (method 2), in the case of vital building and facilities, or any combination of these two methods.

Limited Rehabilitation Objectives

The rehabilitation that addresses the entire building's structural and non-structural systems, but uses a lower seismic hazard or lower target Building Performance Level than the BSO, is termed as *Reduced Rehabilitation Objective*.

The rehabilitation that addresses a portion of the building without rehabilitating the complete lateral-force resisting system is termed *Partial Rehabilitation*. A Partial Rehabilitation will be designed and constructed considering future completion of a Rehabilitation Objective intended to improve the performance of the entire structure.

EC8 part 3 [67] doesn't directly use the name "performance objectives", and specify three limit states LS (related to the building behaviour), associated with three recommended seismic hazard levels. This code gives the freedom to the national authorities to decide whenever all three, two or just one of the LS's must be checked and also leave them the possibility to establish the earthquake hazard associated.

3.2.3. Earthquake hazard level

Earthquake Ground Motion = engineering characteristics of the shaking at the site for a given earthquake or a level of shaking that has a certain probability of occurring [19].

Seismic hazard due to ground shaking shall be based on the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and a selected Earthquake Hazard Level [73].

Seismic hazard due to ground shaking shall be defined as acceleration response spectra or acceleration time-histories on either a probabilistic or deterministic basis.

The analysis and evaluation procedures of FEMA 356 [73] are primarily aimed at improving the performance of buildings under loads and deformations imposed by seismic shaking. However, other seismic hazards could exist at the building site that could damage the building, regardless of its ability to resist ground shaking.

Probabilistic hazards are defined in terms of the probability that more severe demands might be experienced (probability of exceedance) in a 50 year period (see Table 3-2) [73].

Deterministic demands are defined within a level of confidence in terms of a specific magnitude event on a particular major active fault.

FEMA 356 [73] defines two basic Earthquake Hazard Levels:

- Basic Safety Earthquake 1 (BSE-1);
- Basic Safety Earthquake 2 (BSE-2).

In addition to the BSE-1 and BSE-2 Earthquake Hazard Levels, Rehabilitation Objectives may be formed considering ground shaking due to Earthquake Hazard Levels with any defined probability of exceedance, or based on any deterministic event on a specific fault.

There appears to be a widespread perception that uniform hazard spectra, derived from probabilistic seismic hazard curves will provide adequate information for performance based design and evaluation. These spectra, which account for the contributions of all seismic sources that may affect the site, are usually not representative for of any earthquake. Although many cases spectral accelerations (or displacements) obtained from these spectra provide adequate information to describe the seismic demands imposed on structures, in many other cases they do not.

Actual time-history records show significant variations in spectral ordinates, and the frequency characteristics of time history records, which control higher mode effects and to some extent the inelastic response of structures, are masked by period specific spectral hazard analysis. Perhaps most important, the effects of pulse-type near-fault ground motions are hidden away in a uniform hazard spectrum. Nowadays it is widely acknowledged that spectra of these ground motions look very different from uniform hazard spectra and that the effects of these motions on the inelastic response of multi-degree of freedom structures cannot be deduced from an elastic response spectrum [114].

Thus, in addition to further refinements in uniform hazard spectra, the need exists to consider separately the effects of near-fault ground motions. This requires the generation of magnitude, distance and directivity dependent near-fault ground motions that can be used for performance evaluation. This, as well as the development of procedures for generating soft-soil ground motions, should be short-term research objectives for PBEE. A long-term research objective should be the development of source mechanism, magnitude, and distance dependent bins of ground motions that will, ultimately, replace the use of spectra for performance evaluation – at least at low performance levels at which significant inelastic response is anticipated. Uniform hazard spectra will then still be useful for conceptual design, but their use for performance evaluation should be phased out in time.

The structural engineering contribution to this research needs to focus on the issue of the most appropriate representation of ground motion for performance evaluation. This issue deserves much attention because performance evaluation is an engineering issue and every effort needs to be made to reduce the uncertainties caused by simplifications in the hazard description. There are many other seismic hazard related issues that contribute to uncertainty and need to be evaluated more accurately, including the ground motion duration effects that affect cumulative damage, basin effects that may be critical for long period structures, and the existence of collateral hazards [114].

In Table 3-2 there are given the medium recurrence interval for frequent, occasional, rare and very rare earthquake according to FEMA 356 [73], SEAOC [193], EC8 [67] (suggested values for earthquake) and P100-3/2003 [171].

Table 3-2 Earthquake hazard level

<table>
<tr><td rowspan="6">Earthquake Hazard Level</td><td rowspan="2">Frequency</td><td colspan="2">FEMA 356 [73]</td><td colspan="2">SEAOC Vision 2000 [193]</td><td colspan="2">EC8-3 [67]</td><td colspan="2">P100-3 [171]</td></tr>
<tr><td>MRI</td><td>PE</td><td>MRI</td><td>PE</td><td>MRI</td><td>PE</td><td>MRI</td><td>PE</td></tr>
<tr><td>Frequent</td><td>72</td><td>50%/50</td><td>43</td><td>50%/30</td><td>-</td><td>-</td><td>30</td><td>63%/50</td></tr>
<tr><td>Occasional</td><td>225</td><td>20%/50</td><td>72</td><td>50%/50</td><td>225</td><td>20%/50</td><td>100</td><td>40%/50</td></tr>
<tr><td>Rare</td><td>474</td><td>10%/50</td><td>475</td><td>10%/50</td><td>475</td><td>10%/50</td><td>475</td><td>10%/50</td></tr>
<tr><td>Very Rare</td><td>2475</td><td>2%/50</td><td>970</td><td>10%/100</td><td>2475</td><td>2%/50</td><td>975</td><td>5%/50</td></tr>
</table>

PE - Probability to exceed; MRI - Medium recurrence interval

3.2.4. Building, structural and nonstructural performance levels

The seismic rehabilitation standards define the Target Building Performance Levels like combinations of the performance levels of both structural and non-structural components.

3.2.4.1. Structural performance level

According to FEMA 356 [73] and ATC40 [19], the Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section.

The discrete Structural Performance Levels are:

- Immediate Occupancy (S-1);
- Life Safety (S-3);
- Collapse Prevention or Structural Stability (ATC) (S-5) ;
- Not Considered (S-6).

Design procedures and acceptance criteria corresponding to these Structural Performance Levels are specified in dedicated standards.

The intermediate Structural Performance Ranges are the:

- The Damage Control Range (S-2);
- The Limited Safety Range (S-4).

Acceptance criteria for intermediate structural performance range shall be obtained by interpolating.

Structural Performance Level S-1, Immediate Occupancy, imply the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

Design for the **Damage Control Structural Performance Range (S-2)** may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents, or to preserve important historic features when the cost of design for immediate occupancy is high.

Structural Performance Level, Life Safety (S-3), imply the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

Structural Performance, Limited Safety (S-4) shall be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

Structural Performance Level Collapse Prevention (S-5) means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force resisting system, large permanent lateral deformation of the structure, and — to a more limited extent — degradation in vertical-load-carrying capacity. However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

A building rehabilitation that does not address the performance of the structure shall be classified as **Structural Performance Not Considered (S-6).**

To establish the Structural Performance Level, values for drifts are intended to be qualitative descriptions of the approximate behaviour of structures meeting the indicated levels (see Table 3-3).

Table 3-3 Structural Performance Levels - Vertical Elements [73]

Elements	*Collapse Prevention*	*Life Safety*	*Immediate Occupancy*
Concrete Frames	*4% transient or permanent*	*2% transient; 1% permanent*	*1% transient; negligible permanent*
Steel Moment Frames	*5% transient or permanent*	*2.5% transient; 1% permanent*	*0.7% transient; negligible permanent*
Braced Steel Frames	*2% transient or permanent*	*1.5% transient; 0.5% permanent*	*0.5% transient; negligible permanent*
Concrete Walls	*2% transient or permanent*	*1% transient; 0.5% permanent*	*0.5% transient; negligible permanent*
Unreinforced Masonry Infill Walls	*0.6% transient or permanent*	*0.5% transient; 0.3% permanent*	*0.1% transient; negligible permanent*
Unreinforced Masonry (Noninfill) Walls	*1% transient or permanent*	*0.6% transient; 0.6% permanent*	*0.3% transient; 0.3% permanent*
Reinforced Masonry Walls	*1.5% transient or permanent*	*0.6% transient; 0.6% permanent*	*0.2% transient; 0.2% permanent*

3.2.4.2. Non-structural performance level

FEMA 356 [73] and ATC 40 [19], based on standard criteria, offer detailed conditions to accomplish the Non-structural Performance Levels of a building. The following Non-structural (N) Performance Levels are considered:

- Operational (N-A);
- Immediate Occupancy (N-B);
- Life Safety (N-C);
- Hazards Reduced (N-D);
- Not Considered (N-E).

Standards offer Nonstructural Performance Levels and Damage limitations for:

- *Architectural Components* (cladding, glazing, partitions, ceilings, parapets and ornamentation, canopies and marquees, chimneys and stacks, stairs and fire escapes, doors);
- *Mechanical, Electrical, and Plumbing Systems/Components* (elevators, HVAC equipment, manufacturing equipment, ducts, piping, fire sprinkler system, fire alarm system, emergency lighting, electrical distribution equipment, light fixtures, plumbing);
- *Contents* (computer systems, desktop equipment, file cabinets, book shelves, hazardous materials, art objects).

3.2.4.3. Building performance level

Building Performance Level = Structural Performance Level + Non-structural Performance Level [19].

Once the performance level for structural and non-structural elements have been establish, the designer could decide on the target building performance level. Table 3-4 present the building performance levels and the relation between them according to different standards.

Table 3-4 Building performance level

Standard	Vision 2000 [193]	NEHRP / FEMA 356 [73]	EC8 -3 limit state [67]	P100 – 3 [171]
Building performance level	Fully Functional	Operational	Limited Damage	Limited Damage
	Operational	Immediate Occupancy		
	Life Safety	Life Safety	Severe Damage	Life Safety
	Near Collapse	Collapse Prevention	Near Collapse	Collapse Prevention

Several common target Building Performance Levels described in this chapter are shown in Figure 3-4. Many combinations are possible as structural performance can be selected at any level in the two Structural Performance Ranges. Table 3-5 indicates the possible combinations of target Building Performance Levels and provides names for those most likely to be selected as the basis for design [73].

Operational Building Performance Level (1-A) Buildings meeting this target Building Performance Level are expected to sustain minimal or no damage to their structural and non-structural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with electri power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this target Building Performance Level pose an extremely low risk to life safety.

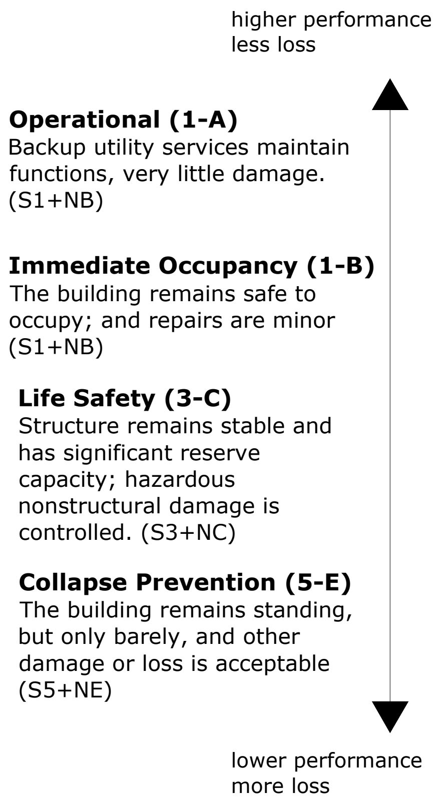


Figure 3-4 Target Building Performance Levels and Ranges [73]

Immediate Occupancy Building Performance Level (1-B)

Buildings meeting this target Building Performance Level are expected to sustain minimal or no damage to their structural elements and only minor damage to their non-structural components. While it would be safe to reoccupy a building meeting this target Building Performance Level immediately following a major earthquake, non-structural systems may not function, either because of the lack of electrical power or internal damage to the equipment. Therefore, although immediate re-occupancy of the building is possible, it may be necessary to perform some cleanup and repair and await the restoration of utility service before the building can function in a normal mode. The risk to life safety at this target Building Performance Level is very low.

Life Safety Building Performance Level (3-C)

Buildings meeting this level may experience extensive damage to structural and non-structural components. Repairs may be required before re-occupancy of the building occurs, and repair may be deemed economically impractical. The risk to life safety in buildings meeting this target Building Performance Level is low.

Collapse Prevention Building Performance Level (5-E)

Buildings meeting this target Building Performance Level may pose a significant hazard to life safety resulting from failure of non-structural components. However, because the building itself does not collapse, gross loss of life may well be avoided, but economically, the building is practically lost.

Table 3-5 Target Building Performance Levels and Ranges [73]

		Structural Performance Levels and Ranges					
		S-1 Immediate Occupancy	**S-2 Damage Control Range**	**S-3 Life Safety**	**S-4 Limited Safety Range**	**S-5 Collapse Prevention**	**S-6 Not Considered**
Non-structural Performance Levels	**N-A Operational**	*Operational 1-A*	*2-A*				
	N-B Immediate Occupancy	*Immediate Occupancy 1-B*	*2-B*	*3-B*			
	N-C Life Safety	*1-C*	*2-C*	*Life Safety 3-C*	*4-C*	*5-C*	*6-C*
	N-D Hazards Reduced		*2-D*	*3-D*	*4-D*	*5-D*	*6-D*
	N-E Not Considered				*4-E*	*Collapse Prevention 5-E*	*No rehabilitation*

Damage Control Levels, regarding the structural typology and load bearing elements, are associated with relevant Building Performance Levels as shown in Table 3-6.

Table 3-6 Damage Control and Building Performance Levels [73]

	Target Building Performance Levels			
	Collapse Prevention Level (5-E)	**Life Safety Level (3-C)**	**Immediate Occupancy Level (1-B)**	**Operational Level (1-A)**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load bearing columns and walls function. Large permanent drifts. Some exits blocked. Infill and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load bearing elements function. No out-of plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Non-structural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.

3.2.5. Selection of building performance objectives

The Rehabilitation Objective selected as target for design or evaluation will determine, to a great extent, the cost and feasibility of any rehabilitation project, as well as the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes. Table 3-7 indicates the range of Rehabilitation Objectives that may be used in accordance with FEMA 356 [73] linked to the Earthquake Hazard Level. For comparison in Table 3-8 there is presented the matrix of building performance objectives, provided by Romanian Seismic Rehabilitation Standard P100-3/2003 [171].

Table 3-7 Rehabilitation Objectives [73]

		Target Building Performance Levels			
		Operational Performance	Immediate Occupancy Performance	Life Safety Performance	Collapse Prevention Performance
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	10%/50 year	i	j	k	l
	2%/50 year	m	n	o	p

Notes:

1. Each cell in the above matrix represents a discrete Rehabilitation Objective.
2. The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives, as follows:

- k + p = *Basic Safety Objective* (BSO);
- k + p + any of a, e, i, b, f, j, or n = *Enhanced Objectives*;
- alone or n alone or m alone = *Enhanced Objective*;
- k alone or p alone = *Limited Objectives*;
- c, g, d, h, l = *Limited Objectives*.

Table 3-8 Rehabilitation Objectives (P100-3) [171]

Earthquake Hazard Level	Target Building Performance Levels		
	Limited Damage	Life Safety	Collapse Prevention
30 year	BSO		
50 year	BSE-1 BSE-2	LRO	
100 year		BSO	
225 year		BSE-2	
475 year		BSE-1	
975 year			BSE-1

A series of standard performance objectives, appropriate for the design of different categories of buildings is needed. Such standards would relieve unsophisticated users of the need to make a difficult selection for which they are unprepared [89].

Such a series of standard performance objectives, recommended by Vision 2000, are indicated in Figure 3-5. Each diagonal line in the figure indicates design performance levels and earthquakes recommended for the design of buildings of different occupancies and uses. Informed building users of course, could certainly select more stringent performance objectives, if desired. The adoption of such a standard would relieve the design engineer and building user from having to select such a basis.

Earthquake Probability	Performance Objective			
	Fully Operational	Operational	Life Safe	Near Collapse
Frequent	□	Unacceptable		
Occasional	O	□	Performance	
Rare	■	O	□	
Very Rare		■	O	□

□ Basic Facilities; O Essential/Hazardous Emergency Response Facilities; ☐ Safety Critical Facilities

Figure 3-5. SEAOC Vision 2000 Performance Objective [193]

3.2.6. Comments

The innovation brought by the new generation of seismic evaluation standards is the introduction of a clear basis in order to predict the behaviour of the building submitted to an earthquake motion. By understanding the Performance Objectives, the engineer can design the damage levels of structural and non-structural members at a certain intensity of the seismic motion.

The matrix (Table 3-9) shows the correlation between different components of PBE procedure for the case of a fictitious example in order to reach a given objective. For this purpose, one follows the general flow-chart in Figure 3-6.

For a non seismic residential reinforced concrete frame, the Basic Safety Objective has been established by the owner together with the designer. To accomplish this Performance Objective, one uses Table 3-7 and selects two situations k + p to be checked. This means that, at a rare earthquake (474 - 10%/50 year), the building performance level should be in the Life Safety range and for a very rare earthquake (2475 - 2%/50 year) the building performance should be Collapse Prevention. The Building Performance Level oance decided, we should decide on the Performance Levels for Structural and Non-structural members should be chosen (see Table 3-5).

Table 3-9 PBE application example

Objective	Verification	Earthquake hazard (MRI yrs)	Building Performance Level	Structural Performance Level	Nonstructural Performance Level
Basic Safety Objective	k	474	Life Safety (3-C)	Life Safety (S-3)	Life Safety (N-C)
	p	2 475	Collapse Prevention (5-D)	Collapse Prevention (S-5)	Hazards Reduced (N-4)

For the first Building Performance level, i.e. “Life Safety” (see Table 3-9), Life Safety Level was imposed for both structural and nonstructural elements. For the second Building Performance level, i.e “Collapse Prevention”, Collapse Prevention Level is imposed for structural level and Hazards Reduced Level for nonstructural elements, in order to avoid large or heavy items that pose a high risk of falling hazard to a large number of people — such as parapets, cladding panels, heavy plaster ceilings are prevented from falling [73].

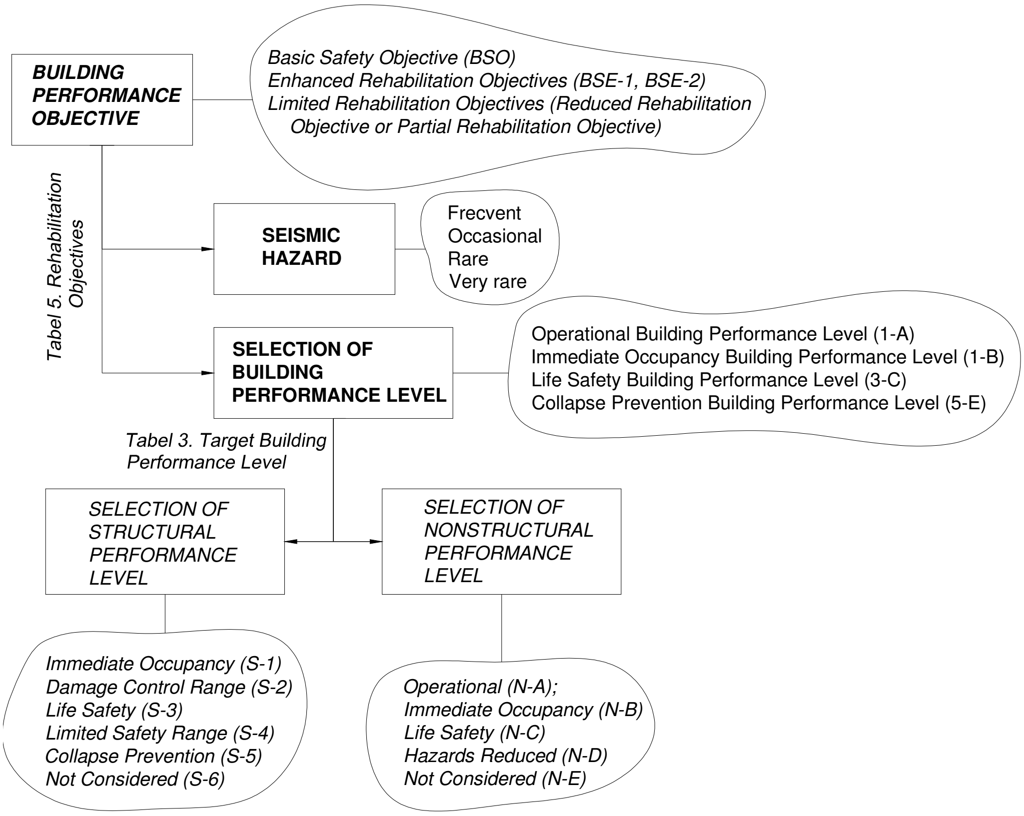


Figure 3-6 Selection of seismic hazard and performance levels for structural and non-structural members

3.3. ANALYSIS METHODS

3.3.1. Introduction

The most important effect of earthquakes on building structures is the inertia forces produced in the building due to ground shaking. Earthquake being a rare event, structures are usually designed to resist earthquake action in the inelastic range of response. Most of the existing structures were not designed for seismic action at all, and are therefore expected to respond beyond the elastic limit under a major earthquake. The dynamic nature of an earthquake action, which has components along the two horizontal directions as well as the vertical one, and the possible inelastic structural response, implies a nonlinear dynamic analysis procedure on a three-dimensional model of the building structure. Though this type of analysis provides the most "exact" modelling of structural response under earthquake action, it requires a high degree of expertise, and can be very time-consuming. In many cases it is possible to adopt more simple analysis procedures.

The simplifications may involve the model of the structure (two plane models instead of a three-dimensional one), time-history response (static analysis instead of dynamic one), and inelastic structural response (linear elastic analysis instead of nonlinear analysis).

There are five generally adopted analysis procedures used for the seismic analysis of structures (FEMA 356 [73]; Eurocode 8 [66]) presented bellow in a hierarchical order:

- lateral force method (linear static procedure);
- response spectrum analysis;
- linear time-history analysis;
- nonlinear static procedure (pushover analysis);
- nonlinear time-history analysis.

The linear procedures maintain the traditional use of a linear stress-strain relationship, but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response. The Nonlinear Static Procedure, often called "pushover analysis," uses simplified nonlinear techniques to estimate seismic structural deformations. The Nonlinear Dynamic Procedure, commonly known as nonlinear time history analysis, requires considerable judgment and experience to perform [200].

The acceptance criteria for the various performance objectives are prescribed for each of the analytical procedures, and numerical values of the acceptance criteria for various structural and nonstructural systems are provided in PBE codes.

Guidance on the global model of the structure and criteria for selection of the analysis procedure are available in seismic design codes ([73]; [66]; [67]). A summary of their requirements is presented hereafter.

3.3.2. Global analysis and modeling requirements

3.3.2.1. General considerations

Due to the dynamic nature of seismic action, the structural model should adequately represent not only the distribution of stiffness, but also the distribution of mass within the structure. When nonlinear analysis methods are used, in addition to stiffness and mass, the global structural model should include the distribution of strength within the structure. [200].

Horizontal torsion. There is no needed to be considered it in structures with flexible floor diaphragms. An accidental eccentricity is introduced to account for uncertainties in the distribution of stiffness and mass, as well as for the rotational components of the ground motion.

A three-dimensional model of the structure accounts directly for torsion due to eccentricity between the centres of mass and stiffness, and needs an explicit consideration of accidental eccentricity only.

Diaphragms. It is generally preferred that floor diaphragms be rigid in their plane. Rigid diaphragms provide a connection between lateral force resisting systems and the gravity load resisting systems within a building, and enable for the different lateral load resisting systems in the building to contribute to the global lateral resistance of the structure [200].

When diaphragms cannot be considered rigid, structural models should account explicitly for the in-plane stiffness of the floor diaphragms.

Second-order effects. When a structure is very flexible under lateral loads, a first-order analysis may substantially underestimate forces and deformation. A second-order analysis is necessary in this case. When a non-linear analysis is used, second-order effects should be considered directly in the formulation of force-deformation relationships for all elements subjected to axial forces.

Displacement analysis. If the structure responds mainly in the elastic range under the design seismic action, lateral displacements can be reliably estimated based on a linear analysis (static or dynamic). However, if the structure is expected to experience significant yielding under the design seismic action, lateral deformations can be significantly larger than the ones estimated based on a linear analysis. The effects that can contribute to inelastic deformations larger than the elastic ones are: (1) frequency content of the ground motion, in relation to the fundamental period of vibration of the building, (2) duration of the ground motion, (3) hysteretic load deformation characteristics of structural elements, including strength and stiffness degradation [200].

Soil-structure interaction. The most important effect of soil-structure interaction is the elongation of the period of vibration of the structure due to the flexibility of the foundation-soil interface. It needs to be considered when the increased period of vibration of the building amplifies spectral accelerations [200].

3.3.2.2. Modeling Parameters and Acceptance Criteria

The acceptability of force and deformation actions shall be evaluated for each component of the building. Prior to selecting component acceptance criteria, each component shall be classified as primary or secondary and each action shall be classified as deformation-controlled (ductile) or force-controlled (fragile). Component strengths, material properties, and component capacities shall be determined. Component acceptance criteria not presented in standards shall be determined by qualification testing [73].

All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level [73].

All actions shall be classified as either deformation-controlled or force-controlled using the component force versus deformation curves shown in Figure 3-7.

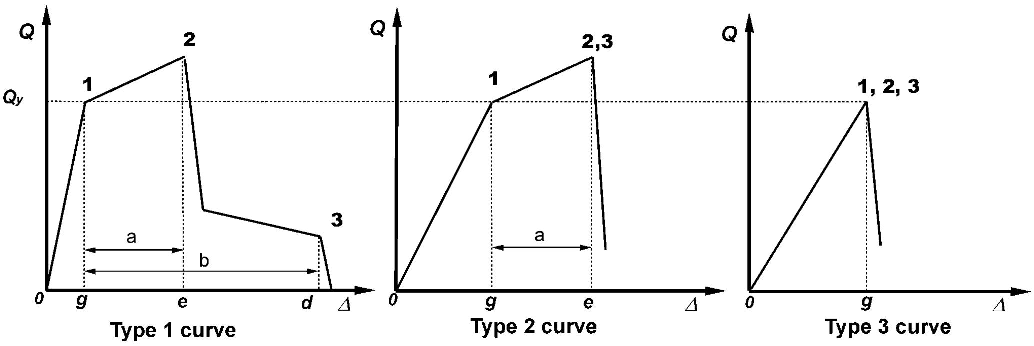
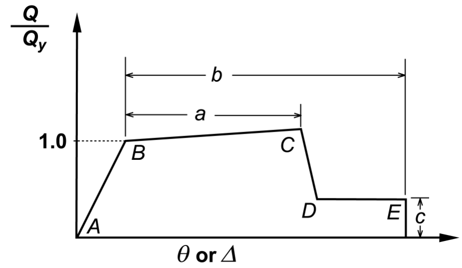


Figure 3-7 Component Force versus Deformation Curves (FEMA 356) [73]

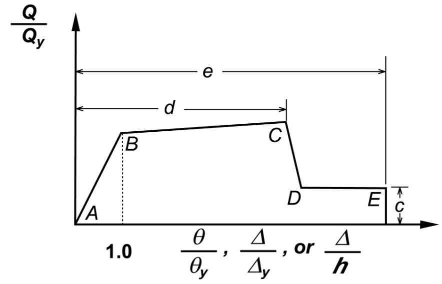
The **Type 1** curve depicted in Figure 3-7 is representative of ductile behaviour where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3. The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3). Primary component actions exhibiting this behaviour shall be classified as deformation-controlled if the strain-hardening or strain softening range is such that e > 2g; otherwise, they shall be classified as force-controlled. Secondary component actions exhibiting Type 1 behaviour shall be classified as deformation-controlled for any e/g ratio [73].

The **Type 2** curve depicted in Figure 3-7 is representative of ductile behaviour where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary and secondary component actions exhibiting this type of behaviour shall be classified as deformation-controlled if the plastic range is such that e > 2g; otherwise, they shall be classified as force controlled [73].

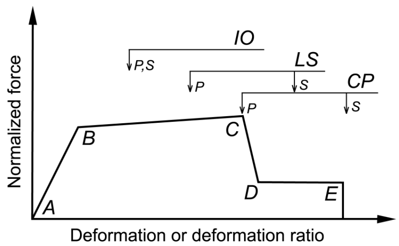
The **Type 3** curve depicted in Figure 3-7 is representative of a brittle or fragile behaviour where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary and secondary component actions displaying Type 3 behaviour shall be classified as force – controlled [73].



(a) Deformation



(b) Deformation ratio



(c) Component or element deformation acceptance criteria

Figure 3-8 Generalized Component Force-Deformation Relations for Depicting Modeling and Acceptance Criteria [73]

For some components it is convenient to prescribe acceptance criteria in terms of deformation (e.g., θ or Δ), while for others it is more convenient to give criteria in terms of deformation ratios. To accommodate this, two types of idealized force vs. deformation curves are used in Figure 3-8 (a) and (b) [73].

Figure 3-8 (a) shows normalized force (Q/Q_{CE}) versus deformation (θ or Δ) and the parameters a, b, and c. Figure 3-8 (b) shows normalized force (Q/Q_{CE}) versus deformation ratio (θ/θ_y, Δ/Δ_y, or Δ/h) and the parameters d, e, and c [73].

Elastic stiffness and values for the parameters a, b, c, d, and e that can be used for modeling components are given. Acceptance criteria for deformation or deformation ratios for primary members (P) and secondary members (S) corresponding to the target Building Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) as shown in Figure 3-8 (c) are given in specific chapters of standards [73].

Structural typologies

The P-BSA of the building is treated separately depending on structural typologies. FEMA 356 [73] provides in the general chapter qualitative definition and in the dedicated chapters provides, quantitative acceptance criteria for ensuring that a specific level of performance be achieved.

Detailed criteria for the calculation of individual component force and deformation capacities shall comply with the requirements in individual materials chapters as follows [73]:

- *Foundations;*
- *Elements and components composed of steel or cast iron:*
    - Steel Moment Frame;
    - Steel Braced Frame;
    - Steel Plate Shear Walls;
    - Steel Frame with Infills;
    - Diaphragm.
- *Elements and components composed of reinforced concrete:*
    - Concrete Moment Frames;
    - Precast Concrete Frames;
    - Concrete Frames with Infills;
    - Concrete Shear Walls;
    - Concrete Braced Frames;
    - Cast-in-place Concrete Diaphragms;
    - Precast Concrete Diaphragms.
- *Elements and components composed of reinforced or unreinforced masonry:*
    - Masonry Walls;
    - Masonry Infills.
- *Elements and components composed of timber, light metal studs, gypsum, or plaster products:*
    - Wood and Light Frame Shear Walls;
    - Wood Diaphragms.
- *Seismic isolation systems and energy dissipation systems;*
- *Nonstructural (architectural, mechanical, and electrical) components;*
- *Elements and components comprising combinations of materials are covered in the Chapters associated with each material.*

For exemplification Generalized Component Force-Deformation Relations for Reinforced Concrete Beams and Masonry Walls Modeling and Acceptance Criteria are presented in Figure 3-9.

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

						Acceptable Drift Ratio (%)[1]				
						Performance Level				
							Component Type			
							Primary		Secondary	
f_{ae} / f_{me}	L / h_{eff}	$\rho_g f_{ye} / f_{me}$	c	d %	e %	IO %	LS %	CP %	LS %	CP %
Wall Components Controlled by Flexure										
0.00	≤ 0.5	0.01	0.5	2.6	5.3	1.0	2.0	2.6	3.9	5.3
		0.05	0.6	1.1	2.2	0.4	0.8	1.1	1.6	2.2
		0.20	0.7	0.5	1.0	0.2	0.4	0.5	0.7	1.0
	1.0	0.01	0.5	2.1	4.1	0.8	1.6	2.1	3.1	4.1
		0.05	0.6	0.8	1.6	0.3	0.6	0.8	1.2	1.6
		0.20	0.7	0.3	0.6	0.1	0.2	0.3	0.5	0.6
	≥ 2.0	0.01	0.5	1.6	3.3	0.6	1.2	1.6	2.5	3.3
		0.05	0.6	0.6	1.3	0.2	0.5	0.6	0.9	1.3
		0.20	0.7	0.2	0.4	0.1	0.2	0.2	0.3	0.4
0.038	≤ 0.5	0.01	0.4	1.0	2.0	0.4	0.8	1.0	1.5	2.0
		0.05	0.5	0.7	1.4	0.3	0.5	0.7	1.0	1.4
		0.20	0.6	0.4	0.9	0.2	0.3	0.4	0.7	0.9
	1.0	0.01	0.4	0.8	1.5	0.3	0.6	0.8	1.1	1.5
		0.05	0.5	0.5	1.0	0.2	0.4	0.5	0.7	1.0
		0.20	0.6	0.3	0.6	0.1	0.2	0.3	0.4	0.6
	≥ 2.0	0.01	0.4	0.6	1.2	0.2	0.4	0.6	0.9	1.2
		0.05	0.5	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.6	0.2	0.4	0.1	0.1	0.2	0.3	0.4
0.075	≤ 0.5	0.01	0.3	0.6	1.2	0.2	0.5	0.6	0.9	1.2
		0.05	0.4	0.5	1.0	0.2	0.4	0.5	0.8	1.0
		0.20	0.5	0.4	0.8	0.1	0.3	0.4	0.6	0.8
	1.0	0.01	0.3	0.4	0.9	0.2	0.3	0.4	0.7	0.9
		0.05	0.4	0.4	0.7	0.1	0.3	0.4	0.5	0.7
		0.20	0.5	0.2	0.5	0.1	0.2	0.2	0.4	0.5
	≥ 2.0	0.01	0.3	0.3	0.7	0.1	0.2	0.3	0.5	0.7
		0.05	0.4	0.3	0.5	0.1	0.2	0.3	0.4	0.5
		0.20	0.5	0.2	0.3	0.1	0.1	0.2	0.2	0.3
Wall Components Controlled by Shear										
All cases[2]	All cases[2]	All cases[2]	0.4	0.75	2.0	0.4	0.6	0.75	0.75	1.5

Figure 3-9 Modeling Parameters and Acceptance Criteria for Masonry walls controlled by flexure [73]

In the case of reinforced concrete and masonry wall structures a specific guide in order to evaluate and asses the seismic damage and behaviour of an building is presented in FEMA 306 EVALUATION OF EARTHQUAKE DAMAGED CONCRETE AND MASONRY WALL BUILDINGS, Basic Procedures Manual [16] and FEMA 307 EVALUATION OF EARTHQUAKE DAMAGEDCONCRETE AND MASONRY WALL BUILDINGS Technical Resources based on FEMA 273 specification but more oriented on masonry and RC walls buildings. These documents adapt the existing state of knowledge rather than develop completely new techniques. The aim of these documents is to improve the application of the existing knowledge and techniques by using observations of earthquake damage to calibrate analytical models of component behaviour.

Unreinforced Masonry Wall component

URM wall elements can be subdivided, according to FEMA 306 [70], into five Component Types as shown in Figure 3-10, based on the mode of inelastic behaviour. Figure 3-10 also shows some of the common behaviour modes. The majority of modes relate to in-plane damage, but out-of-plane damage can occur as well in each of the systems, often in combination with in-plane damage. The five component types are described bellow [70] .

- URM1: Solid cantilever walls. Such walls are typically found adjacent to other buildings or on alleys, and they act as cantilevers up from the foundation.
- URM2: This component is a weak pier in a perforated wall. In this system, inelastic deformation occurs in the piers.
- URM3: This component is a weak spandrel in a perforated wall. Inelastic deformation occurs first in the spandrels, which may create multi-storey piers similar to URM1 or URM2 and then lead to inelastic deformation and damage in the piers.
- URM4: This component is a strong spandrel in a weak pier-strong spandrel mechanism. By definition, it should not suffer damage, and it is not discussed further in the repport.
- URM5: Perforated wall with panel zone weak joints. Inelastic deformation occurs in the region where the pier and spandrel intersect. Such damage is not generally observed in experimental tests, nor is it seen in actual earthquakes, except at outer piers of upper stories. In this document, such damage is considered a case of corner damage and, when caused by in-plane demands, is addressed as part of the URM3 spandrel provisions.

For each type of components, the FEMA 306 documents present in detail the ductility category, the behaviour mode, likelihood of occurrence and damage guide reference. There is also mentioned for each "component damage", how to distinguish the failure mechanism by observation and by analysis, offering clear modelling for evaluation procedures and acceptance criteria for both un-damaged and damaged elements (see Figure 3-11), depending on a clear defined level of damage. All the failure mechanisms are presented together with performance restoration typical measures. In FEMA 307 [16], all the λ - modification factors are sustained by available experimental tests and it contains a brief description of the key technical aspects that address specific masonry component behaviour.

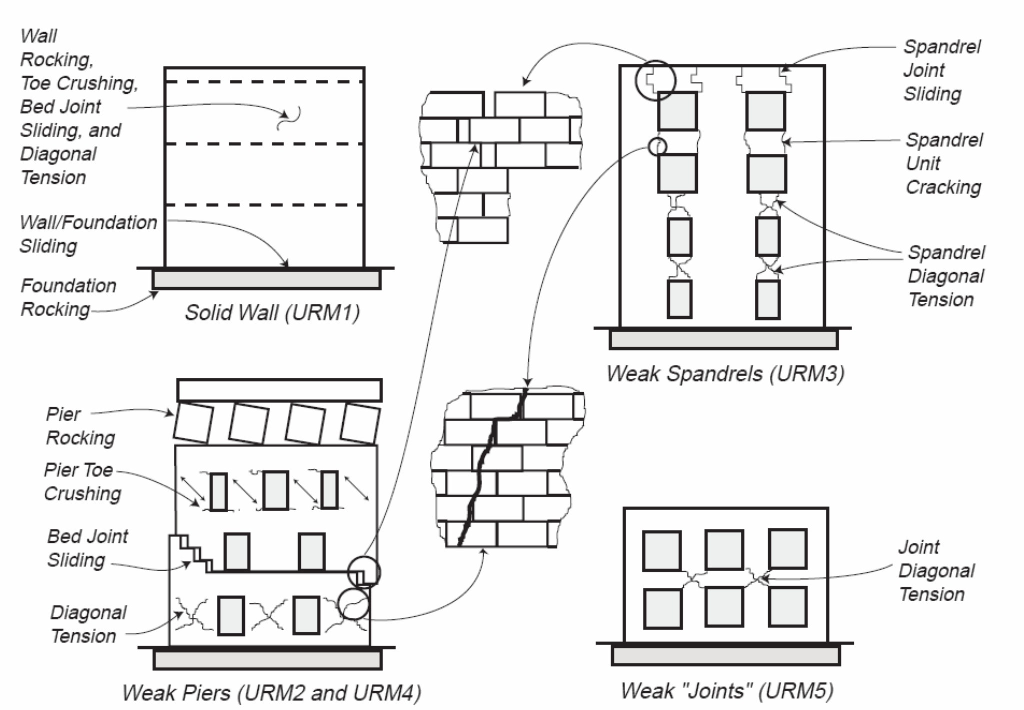


Figure 3-10 URM Walls Components FEMA 306 [70]

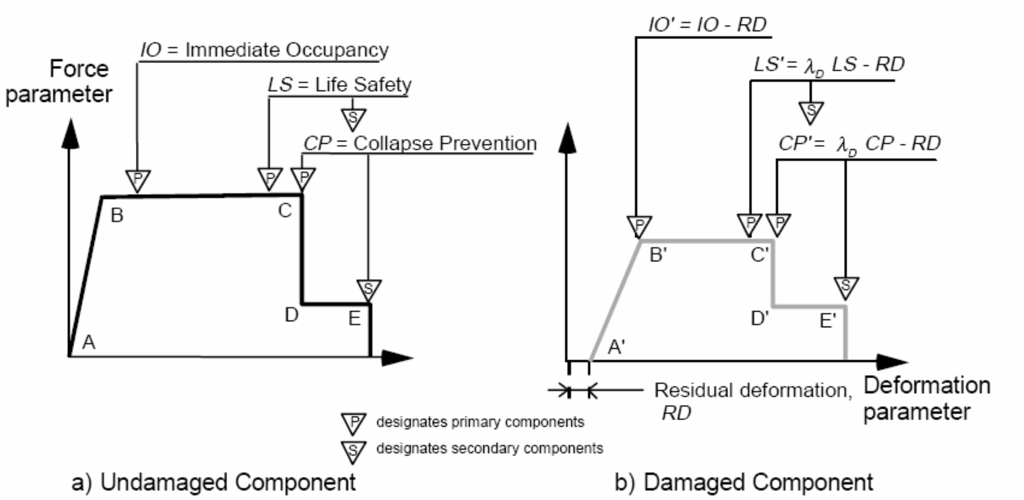


Figure 3-11 Component Modeling and Acceptability Criteria for (a) undamaged and (b) damaged component [70]

3.3.2.3. Linear – Elastic Analysis

Lateral force method

For the assessment of the existing structure (FEMA 356, [73], Eurocode 8-3, [67]), lateral forces are determined based on the elastic response spectrum, and not on the design one (reduced by the behaviour factor q). This procedure intends to estimate the design lateral displacements of the structure rather than the design forces in structural elements, because displacements are better indicators of damage to the structure in the inelastic range than forces [200].

Modal response spectrum and linear time-history

The response spectrum procedure is a generalization of the lateral force method, accounting for more than one mode of vibration in determining the seismic response of the structure.

The response spectrum analysis provides an envelope of displacements and internal forces. When the time-history response is of interest, linear time-history analysis is employed. [200].

When used for the assessment of existing structures, both procedures are intended to provide an estimate of design displacements rather than design forces (FEMA 356, [73]; Eurocode 8-3, [67]).

Acceptance criteria for linear analysis

If linear procedures are used, capacities for deformation-controlled actions shall be defined as the product of m-factors (modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action) or q-factor, and expected strengths, Q_{CE}. Capacities for force-controlled actions shall be defined as lower-bound strengths, Q_{CL} [73].

Deformation-controlled design actions shall be calculated in accordance with [73]:

$$Q_{UD} = Q_G \pm Q_E \tag{3}$$

Where Q_E = action due to design earthquake loads calculated using elastic analysis methods; Q_G = Action due to design gravity loads [73].

Deformation-controlled actions in primary and secondary components and elements shall satisfy the following equation [73]:

$$m \cdot \kappa \cdot Q_{CE} > Q_{UD} \tag{4}$$

Where m = component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level. M-factors are specified in dedicated chapters; Q_{CE} = expected strength of the component or element at the deformation level under consideration for deformation-controlled actions; κ = knowledge factor taken according to knowledge level [73].

Force-controlled actions in primary and secondary components and elements shall satisfy the following equation [73]:

$$\kappa \cdot Q_{CL} > Q_{UF} \tag{5}$$

Q_{CL} = lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions.

Force-controlled design actions Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering load to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building. Alternatively Q_{UF} can be determined as [73]:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \tag{6}$$

Displacement amplifiers, C_1, C_2, and C_3 are divided out when seeking an estimate of the force level present in a component when the building is responding inelastically. J = a coefficient used in linear procedures in order to estimate the actual forces delivered to force-controlled components by other (yielding) components [73].

3.3.2.4. Non-linear Analysis

Static – Pushover

Nonlinear static analysis is usually used together with different procedures (e.g. coefficient method, capacity spectrum method - FEMA 356 [73]; or the N2 method - Eurocode 8-1, [66]) in order to estimate the target displacement under the design seismic action.

Considering that target displacement is intended to represent the maximum displacement experienced during seismic action, and that element inelastic response is modeled directly, nonlinear static procedure will provide reasonable estimates of both displacements and internal forces [200].

Dynamic – Time-history

Nonlinear time-history analysis represents the most advanced method of analysis for the evaluation of the seismic response of structures. Nonlinear time-history analysis provides reasonable estimates of both displacements and internal forces in structural elements [200].

Acceptance criteria for nonlinear analysis

If nonlinear procedures are used, component capacities for deformation-controlled actions shall be taken as permissible inelastic deformation limits, and component capacities for force-controlled actions shall be taken as lower-bound strengths, Q_{CL} [73].

3.3.3. Choice of analysis procedure

3.3.3.1. Knowledge factor

Data on the as-built condition of the structure, components, site, and adjacent buildings shall be collected in sufficient detail in order to perform the selected analysis procedure. The extent of data collected (material properties, initial project drawings, condition assessment and additional information obtained by testing) shall be consistent with minimum, usual, or comprehensive levels of knowledge. Depending on the level of knowledge, there shall be determined the selected Rehabilitation Objective and the analysis procedure in accordance with Table 3-10 [73].

Table 3-10 Data Collection Requirements (FEMA 356) [73]

	Level of knowledge		
	Minimum	Usual	Comprehensive
Rehabilitation Objective	*BSO or Lower*	*BSO or Enhanced*	*Enhanced*
Analysis Procedures	*LSP, LDP*	*All*	*All*

LSP – linear static procedure
LDP – linear dynamic procedure

3.3.3.2. Requirements for analysis procedure selection

Lateral force method

In the elastic range, the dynamic response is governed by the fundamental mode of vibration if the structure is regular in elevation and is not very flexible. The second requirement is expressed in Eurocode 8-1 [65] by limitation of the fundamental period of vibration of structures that can be analyzed by using the lateral force method to the lesser of $4T_C$ and 2 seconds (where T_C is the limit between the constant acceleration and constant velocity area of the spectrum) [200].

Response spectrum and linear time-history analyses

Response spectrum and linear time-history analyses suffer from the drawbacks of linear (elastic) analysis, when applied to highly inelastic structural response. Therefore, these analysis procedures are still not adequate when inelastic demands are non-uniform within the structure and when the structural response is expected to be highly inelastic [200].

Nonlinear static procedure

It is subjected to several limitations, due to the fact that it relies on the assumption that structural response is governed by the fundamental mode shape, and that this shape does not change when the structure yields under increasing lateral loading. Pushover analysis is mainly applicable to estimating seismic demands on low-rise and medium rise structures in which inelastic demands are uniformly distributed along the height of the structure [42]. Eurocode 8-1 [66] requires at least two lateral force distributions ("modal" and uniform).

Several improved procedures based on pushover analysis were proposed by different researchers, in order to account for the influence of higher modes of vibration and the change in the distribution of lateral forces as a result of the change in dynamic properties of the structure as a result of yielding.

Nonlinear time-history analysis

The nonlinear dynamic analysis offers the most "correct/accurate" evaluation of the seismic response of a structure, and can be applied in all cases. However, it requires the greatest degree of expertise of the engineer and the most comprehensive degree of knowledge on the properties of materials and elements.

3.4. CHOICE OF THE INTERVENTION TECHNIQUE

3.4.1. General criteria

Every rehabilitation program of buildings aims at
1. Removing of the causes of the continuing deterioration;
2. Better conserving of the building after the work is completed;
3. Improving of the value.

Alternative solutions shall be finally validated in terms of four criteria groups of different nature [219].

- Cultural and/or social values;
- Technical aspects:
  - Reversibility of intervention, Compatibility, Durability, Corrosion, UV resistance, Aging, Creep, Local conditions, Availability of material/device, Technical capability, Quality control;
- Structural aspects:
  - Structural performance (Strength, Stiffness, Ductility, Fatigue), Response to fire, Sensitivity to changes of actions/resistances e.g. seismic action, temperature, fire, soil conditions, Accompanying measures, Technical support (Codification, Recommendations, Technical rules), Installation/Erection e.g. availability/necessity of lifting equipment;
- Economical and sustainability aspects:
  - Costs, Design, Material/Fabrication, Transportation, Erection / Installation / Maintenance, Preparatory works

Technical aspects refer to decisions covering the overall design and the selection of materials and techniques [219].

3.4.2. Structural performance based validation

The choice of one or another strengthening technique is a multi-criteria problem, as previously shown. The designer always has several solutions at his disposal. Finally, one has to select a solution, that best matches, the assembly of *validation* criteria. In fact, the solution will always represent a *rational compromise* among different criteria, because the one criterion based optimisation leads, in general, to an unacceptable choice.

Moreover, we will hereby show as an example the analysis of possible strengthening solutions, by only considering the structural performance.

Hereby we will show, the example of an R.C. frame [201] which is required to be retrofitted in order to enhance both strength and stiffness to resist seismic actions (see Figure 3-12) For this purpose, six different strengthening techniques are being examined. In order to decide which of the six is the appropriate one, we first need to fix the target of intervention in terms of strength, stiffness and ductility.

Starting from the idea that current strengthening interventions, of the type shown in Figure 3-12, increase both the stiffness and lateral load capacity of the RC frame, the judgement can be based on the analysis of the capacity curve. The capacity curve of the strengthened structure, C_s, generally has a higher slope and peak as compared to the capacity curve before strengthening, C_u. In Figure 3-13, a theoretical situation is considered. Due to the increased stiffness, which translates into a decreased fundamental period, the seismic demand on the structure is also increased, as shown by the demand curve for the strengthened structure, D_s, compared to the one for the unstrengthened structure, D_u. Although the capacity increase is partly alleviated by the increase in seismic demand, the overall performance of the structure is improved as shown by the locations of the performance points on the spectral displacement axis before and after strengthening [35].

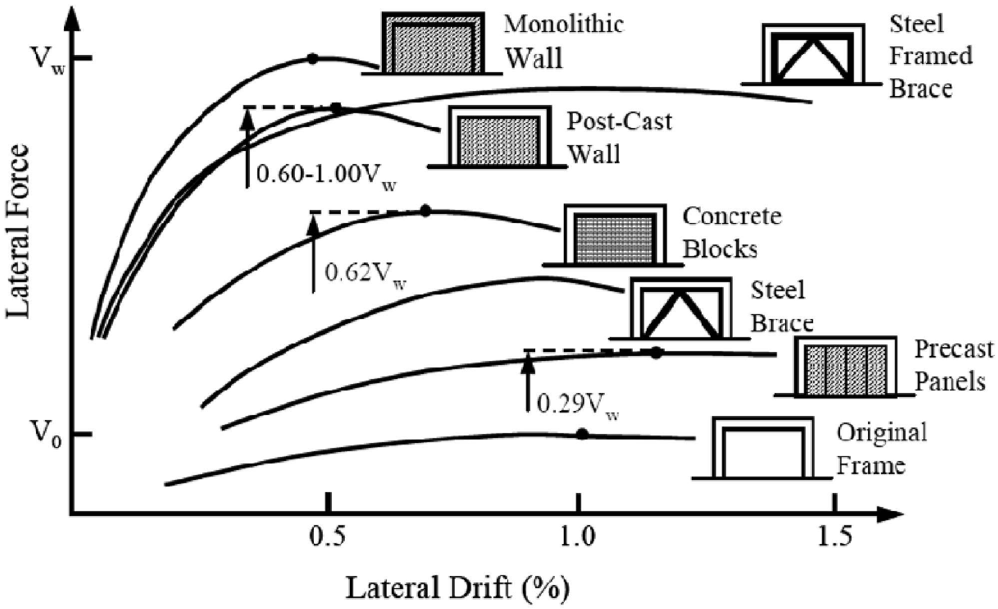


Figure 3-12 Strengthening solutions for a RC Frame (CEB – Fastenings for seismic retrofitting [39]; [201])

Such type of analysis has to be developed for all six solutions in the Figure 3-12. Afterwards, depending on the hierarchy between the demand in strength, stiffness and ductility, and by also considering the other complementary criteria of the previous section (e.g. 3.4.1), the final decision can be taken.

Regarding the structural aspect, the intervention strategy has to make a choice between increasing the strength of the building or enhancing the deformation capacity (e.g. ductility) or a good balance of both. The attempt to increase the resistance leads in most of the cases to a significant increase in stiffness and consequently to increasing seismic force and demands. Anyway, the major problem of masonry is the deformation capacity, thus in order to enhance of deformation capacity of masonry (see Figure 3-13c) to getting more dissipation seems to be the most suitable solution.

Modern retrofitting strategies insist on using of mixed and reversible technologies. The use of "mixed material based technology" enables the optimisation of the performance of retrofitted structures. The reversibility is very important because it offers the possibility to remove a solution when more advanced technologies (or more financial support) become available. This aspect cannot be neglected in case of the buildings of cultural or historical value.

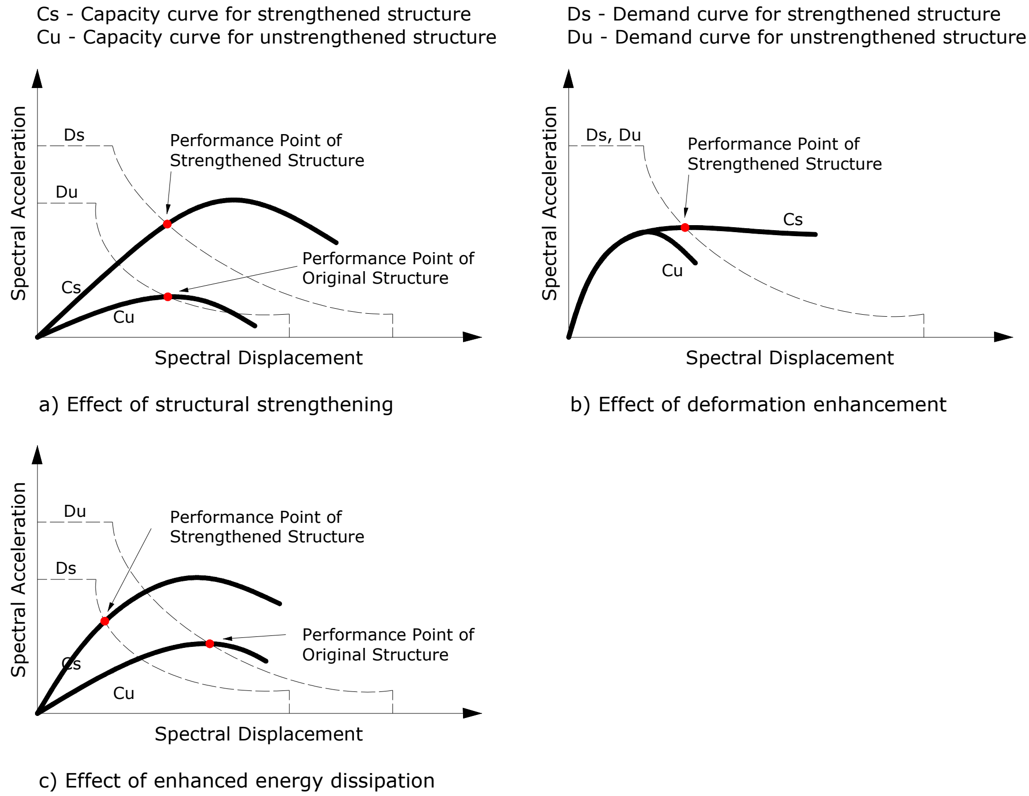


Figure 3-13 Analysis of the concept of strengthening solutions [19][34]

From the statements made above, we may concludes, that for masonry buildings, there is needed a reversible mixed technique able to upgrade resistance and enhance the deformation capacity of masonry shear walls without significant changes in stiffness.

In Table 3-11 there is proposed the following Decisional Matrix for selection and validation of the rehabilitation method.

Table 3-11 Decisional matrix

Structural aspects	L	M	H	Mark
Capability to achieve the requested performance objective (after building evaluation!)				
Compatibility with the structural system (no need for complementary strengthening or confinement measures)				
Adaptability to change of seismic actions typology (near field, far field, T<>Tic, etc)				
Adaptability to change of building typology				
Technical aspects	L	M	H	Mark
Reversibility of intervention				
Durability				
Operational				
Functionally and aesthetically compatible and complementary with the existing building				
Sustainability				
Technical capability				
Technical support (Codification, Recommendations, Technical rules)				
Availability of material/device				
Quality control				
Economical aspects	L	M	H	Mark
Costs (Material/Fabrication, Transportation, Erection, Installation, Maintenance, Preparatory works)				

<u>Legend</u>
L = low, M = medium, H = high
Mark – L (5-6), M (7-8), H (9-10)

3.5. P-BSA METHODOLOGIES

3.5.1. Review of the main evaluation methods

The structural engineering community has developed a new generation of design and seismic evaluation procedures that incorporates performance-based engineering concepts. In a short term, the most appropriate approach seems to be a combination of the nonlinear static (pushover) analysis and the response spectrum approach [224].

Examples of such approach are:

- Capacity spectrum method (CSM), applied in:
    - ATC 40 (Seismic Evaluation and Retrofit of Concrete Buildings, 1996) [19]

    - U.S. Army Corps of Engineers, Technical Manuals (Seismic Design for Buildings and Seismic Design Guidelines for Upgrading Existing Buildings, 1998)
    - Japanese Building Standard Law (BSL 2000)
- Nonlinear static procedure, applied in:
    - FEMA 356 (Prestandard and Commentary for the Seismic Rehabilitation of Buildings, 2000) [73],
    - N2 method developed at the University of Ljubljana [68] and implemented in the draft Eurocode 8 (Design of structures for earthquake resistance, 2001),
    - Modal Pushover Analysis [41]

All methods combine the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. Inelastic spectra or elastic spectra with equivalent damping and period are applied. As an alternative representation of inelastic spectrum, the Yield Point Spectrum has been proposed [22]. Some other simplified procedures based on deformation-controlled design have been developed, e.g. the approaches developed by Priestley [185] and by Panagiotakos and Fardis [174].

The essential difference is related to the determination of the displacement demand (target displacement). If an equivalent elastic spectrum is used, displacement demand is determined based on equivalent stiffness and equivalent damping, that depend on the target displacement and, consequently, iteration is needed. The quantitative values of equivalent damping, suggested by different authors, differ considerably. On the other side, for the methods using inelastic spectra, bilinear idealization of the pushover curve is required. If the bilinear idealization depends on the displacement demand, then the computational procedure becomes iterative too. The procedures also differ in the assumed lateral load pattern, used in pushover analysis, and in the displacement shape, used for the transformation from the MDOF to the SDOF system (and vice versa). Only if the two vectors are related, i.e. if the lateral load pattern is determined from the assumed displacement shape, the transformation from the MDOF to the SDOF system is based on a mathematical derivation [224].

Related to the organization of the evaluation procedure or design for retrofitting a given structure the following general items are emphasized [76]:

- The role of the displacement in the design process
    - Deformation – calculation based (DCB)
    - Iterative deformation – specification based (IDSB)
    - Direct deformation – specification based (DDSB)
- Type of analysis used in the design process
    - Response spectra – initial stiffness based
    - Response spectra – secant stiffness based
    - Time history analysis based
- Structural type limitations
- Limit-state or performance objectives limitations

The matrix in Table 3-12 summarizes the various design procedures that may be applied:

Table 3-12 Matrix of design procedures [76]

	Deformation – calculation based (DCB)	Iterative deformation – specification based (IDSB)	Direct deformation – specification based (DDSB)
Response spectra – initial stiffness	*Moehle [163]* *FEMA 356 [73]* *UBC [215]* *Panagiotakos & Fardis [174]* *Albanesi [12]* *Fajfar [68]*	*Browning [31]*	*SEAOC [192]* *Aschheim & Black [22]* *Chopra & Goel [41]*
Response spectra – secant stiffness	*Freeman [79]* *ATC [20]* *Paret [181]* *Chopra & Goel [40]*	*Gulkan & Sozen [87]*	*Kowalsky [113]* *SEAOC [192]* *Priestley & Kowalsky [185]*
Time history analysis	*Kappos & Manafpour [107]*	*N/A*	*N/A*

The Approximate analysis requires basic structural information in addition to visual screening methodology such as the dimensions of columns, beams and shear walls, which can be determined from building drawings or measurements, usually on the ground floor. Where building drawings are not available, minimum reinforcement is assumed in the structural elements. Concrete strength is usually assumed as a conservative value, however, on site (e.g. Windsor probe) or laboratory measurement of concrete strength is more appropriate for buildings in areas known for variability in material properties. The lateral seismic design loads on the building are calculated by using the static equivalent load method and distributed to the floors according to seismic codes [34]. The calculated load demand is compared to the lateral load capacity of the floor determined either individually for each member, or as a whole by simplifying the building system to one of the forms shown in Figure 3-14.

The former requires the distribution of the floor load to members according to their rigidities. The evaluation of the building is performed by means of a seismic index, I_s, determined by a ratio between the total allowable lateral load and the probable lateral seismic load demand, given by:

$$I_s = \frac{V_{all}}{V} \tag{7}$$

This evaluation is generally performed for ground floor only for savings in time and labour. If in case it is performed for each floor, the most critical index is assigned for the building. A significant advantage of approximate structural evaluation methodologies, other than considerable time savings compared to detailed analysis methods, is the ability to perform a first level prioritization, based on the level of lateral load resistance, for a detailed analysis or retrofit application [34].

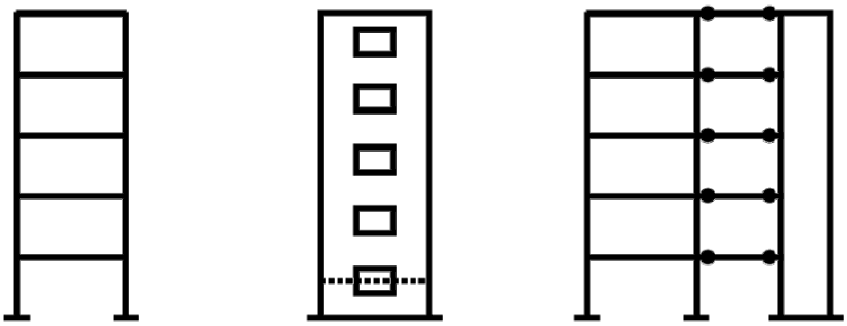
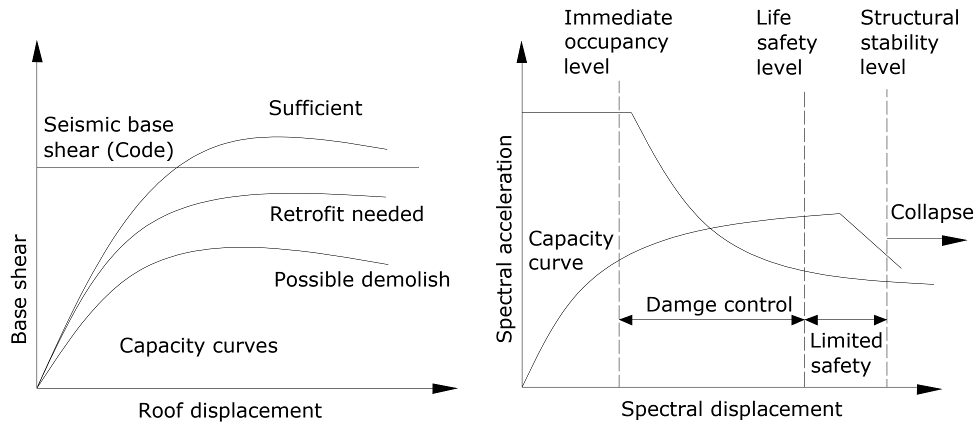


Figure 3-14 Simplified equivalent building systems for the approximate analysis [34]

The detailed evaluation through linear analysis methods is the most commonly used approach since most seismic codes (e.g. [204], [103]) require use of these methods. Based on detailed structural information, member forces under design loads are determined and compared to their ultimate strength. With this methodology, it is possible to accurately determine the overstressed members under design loads; however, it is difficult to assess the seismic risk of the building at the system level. Thus, although this method is useful in prioritizing deficient structures, it may not yield sufficient information needed for determining the optimum retrofit strategies. The current trend is to use the nonlinear analysis techniques, which require approximately the same amount of data, but more engineering effort and expertise compared to the approaches based on linear analysis techniques.

The detailed evaluation using nonlinear analysis provides the most accurate and reliable risk assessment, loss estimation, and retrofit optimization practices at the expense of detailed site, structural, and material information, longer computation times, and a higher level of technical expertise. The linear analysis methodology described above is an integral part of this methodology. By considering the nonlinear inelastic behaviour of structural members under increasing loads, this methodology can predict the nonlinear behaviour of the structural system much more realistically compared to linear analysis techniques [34].

Determining the nonlinear structural behaviour allows for performance-based design, which results in significant savings in seismic retrofit applications ([19], [73]). Figure 3-15(a) shows the typical top displacement vs. the base shear curve obtained from nonlinear pushover analysis of buildings. Using this curve alone, one can perform a preliminary evaluation of the structure's seismic safety by comparing its capacity with the seismic demand determined with the use of the equivalent static load method described in seismic codes. A better performance evaluation can be performed by converting both the capacity curve and the seismic demand spectrum to the acceleration-displacement response spectrum (ADRS) format formed as a relationship between the spectral displacement and the spectral acceleration as shown in Figure 3-15(b). A further improved evaluation can be achieved by obtaining a reduced inelastic response spectrum for the seismic demand to consider the increased damping due to inelastic deformations in the building [19].



a) Evaluation based on equivalent static load b) Evaluation based on performance level

Figure 3-15 Seismic safety evaluation of buildings using nonlinear analysis [34] [19]

The intersection of the capacity and demand curves shown in Figure 3-15 (b) is called the ***performance point of the building***. Based on the location of this performance point, performance level of the building is determined. The intervals of spectral displacement that correspond to different Performance Levels are in principle shown in Figure 3-15(b) and the limits of the performance levels, that are expressed in terms of *interstory drift values*, are recommended in Table 3-3.

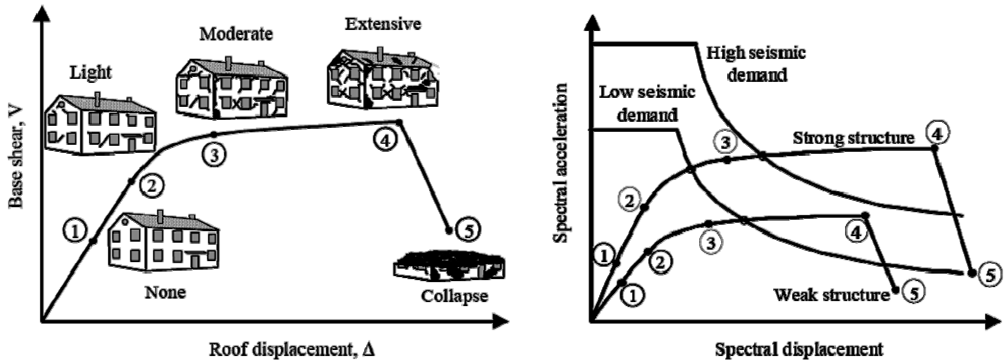
If the performance point is located in the initial portion of the capacity curve where the inelastic deformations are not significant the performance level of the building is *Immediate Occupancy*, which is self explanatory.

For interstory drift values, corresponding to the range of Immediate Occupancy and Life Safety levels, respectively, the performance level of the building is *Life Safety* (or Damage Control). In this region, inelastic deformations are expected in the building that poses no significant threat to the stability of the building and the safety of its occupants. Between the Life Safety and Collapse Prevention (or Structural Stability) Levels, the building performance level is described as *Collapse Prevention* (Limited Safety). Large inelastic deformations are expected which may result in excessive cracking and failure of some structural members, which may pose threat to occupants or result in local failures. Beyond the Collapse Prevention (or Structural Stability) level, the collapse of the building is imminent.

From this discussion, it is apparent that nonlinear analysis is a very convenient methodology for development of realistic fragility curves [34].

3.5.2. Vulnerability Analysis

Vulnerability can simply be defined as the sensitivity of the exposure to seismic hazard(s). The vulnerability of an element is usually expressed as a percentage loss (or as a value between zero and one) for a given hazard severity level [43]. In a large number of elements, like building stocks, vulnerability may be defined in terms of the damage potential to a class of similar structures subjected to a given seismic hazard. Vulnerability analysis reveals the damageability of the structure(s) under varying intensity or magnitudes of ground motion. Multiple damage states are typically considered in the analysis [34].

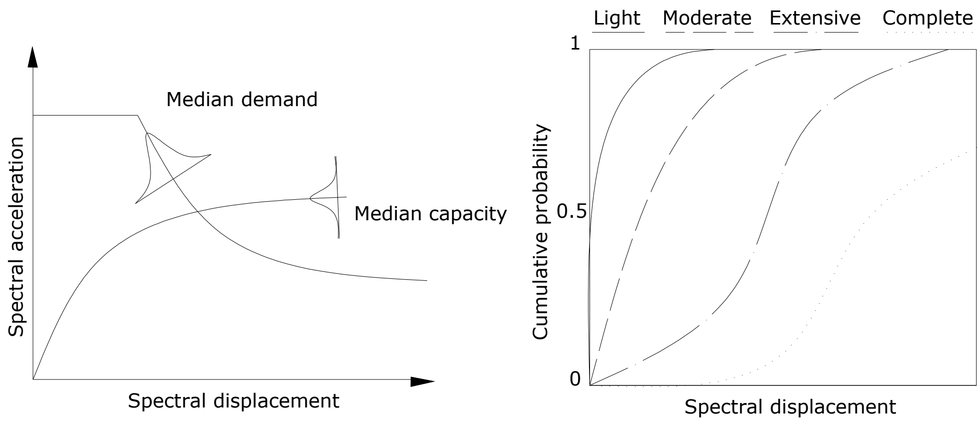


(a) Damage states shown on V-Δ curve (b) Damage levels based on seismic demand

Figure 3-16 Structural vulnerability and damage states for various level of seismic demand [34]

Figure 3-16(a) shows the damage states of a building based on the applied base shear, which can be determined as a function of the seismic demand. The roof displacement – the base shear curve, also called the capacity curve, shown in this figure represents the nonlinear behaviour of a building under increasing load or displacement demand. The damage state of the building varies between none to collapse under increasing levels of demand, which is graphically illustrated in Figure 3-16(a). A relatively more convenient representation of the damage states is provided in Figure 3-16(b) by overlaying both building capacity and seismic demand curves on a different set of axes showing spectral displacement vs. spectral acceleration. Two different capacity and seismic demand curves are shown in the Figure 3-16. The intersection of the capacity and demand curves represents the damage state likely to be experienced by the structure. As can be seen from the figure, the strong structure is likely to suffer from light to moderate damage due to the low seismic demand, and moderate to extensive damage due to the high seismic demand. On the other hand, the weak structure is expected to suffer from moderate to extensive damage due to low seismic demand, and collapse during the high seismic demand due to insufficient seismic resistance [34].

Methods of vulnerability analysis vary based on the exposure information and the complexity of the approach. The vulnerability of structures to ground motion effects is often expressed in terms of fragility curves or damage functions that take into account the uncertainties in the seismic demand and capacity. Fragility functions can be developed for buildings or their components depending on how detailed the risk analysis is performed. Early forms of fragility curves were developed as a function of qualitative ground motion intensities largely based on expert opinion. Recent developments in nonlinear structural analysis have enabled the development of fragility curves as a function of spectral parameters quantitatively related to the magnitude of ground motion. Figure 3-17(a) shows the typical seismic demand and structural capacity curves together with their uncertainties expressed in terms of probabilistic distributions. Based on these curves and the associated uncertainties, the fragility curves shown in Figure 3-17(b) can be constructed for various damage states. Since each damage level is associated with a repair/replacement cost, the probabilistic estimates of the total cost can be estimated using these curves once the hazard is known. This can be achieved by the use of predefined representative fragility curves developed for structures in the same class, or custom damage curves developed through nonlinear analysis of individual structures [34].



a) Uncertainties in seismic capacity and demand b) Fragility curves for various damage states

Figure 3-17 Uncertainties in seismic performance and use of fragility curves [34]

The construction of the fragility or damage curves is the key element in estimating the probability of various damage states in buildings or building components as a function of the magnitude of a seismic event. Thus, the development of realistic fragility curves for the building stock and lifelines in a seismic region constitutes an essential part of a meaningful seismic risk analysis [34].

One of the best known methodologies for assessing the fragility function is HAZUS. This software is based on a methodology for estimating potential earthquake losses on a regional basis, developed under the coordination of the National Institute of Building Science (NIBS) under a cooperative agreement with the Federal Emergency Management Agency (FEMA).

(1) Selection of scenario earthquakes and PESH inputs

(2) Selection of appropriate methods (modules) to meet different user needs

(3) Collection of required inventory data, i.e., how to obtain necessary information

(4) Costs associated with inventory collection and methodology implementation

(5) Presentation of results including appropriate terminology, etc.

(6) Interpretation of results including consideration of model/data uncertainty.

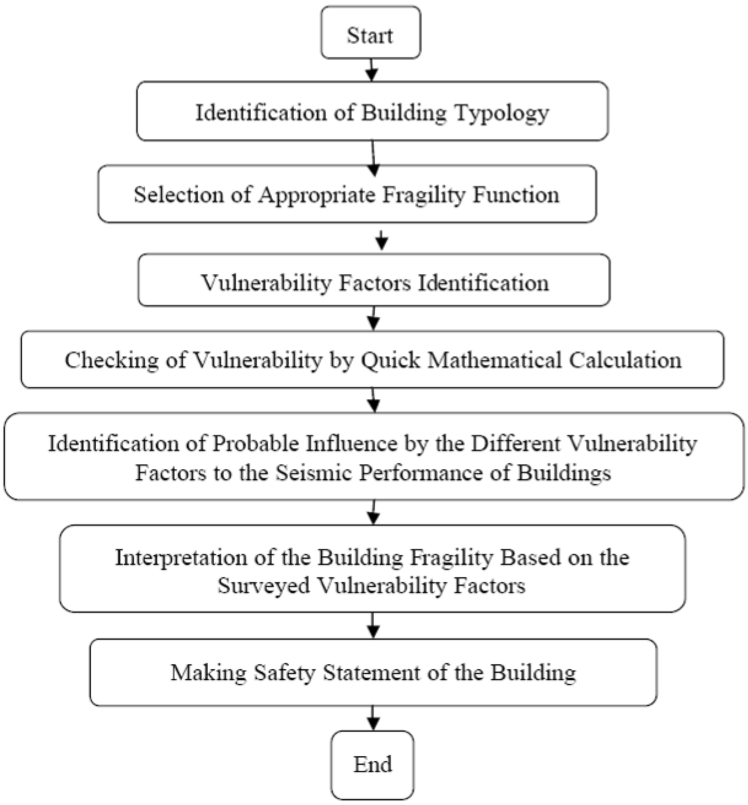


Figure 3-18 Steps evaluation of building safety [88]

3.6. CONCLUDING REMARKS TO APPLY PSBA TO PROPOSED TECHNIQUES

This retrofitting technique attempts to respond at to all major desiderates previously mentioned. Combining metal sheeting, which is resistant and ductile, with masonry, by means of a proper connecting technology, seems to be a suitable solution. The use of "dry" connection easy enables the removal of metallic elements. Prestressed tie connectors provide a very significant confinement effect to masonry. Additionally, the solution offers the advantage of high mechanical properties, e.g. strength and ductility, without changing too much the initial rigidity.

Prior to experimental tests it was believed that this technique can provide a stable post-cracking behaviour to the masonry wall. Moreover, a performance based design methodology can be developed.

There have been presented the main features and performances of a new innovative strengthening technique of masonry which consists in metal sheathing of walls by mild carbon steel or aluminium plates, connected by chemical or prestressed ties. The new technique was validating by tests and show very good behaviour in the range of Life Safety – Collapse Prevention performance levels.

As it has been stated above, an intervention strategy has to choose between increasing the strength and enhancing the deformation capacity (e.g. ductility). The attempt to increase the resistance also leads to secondary undesired negative effects and the major problem of masonry is the deformation capacity. We can be conclude that in case of masonry, the most suitable solution is to enhance the deformation capacity (see Figure 3-19) in order to get more dissipation.

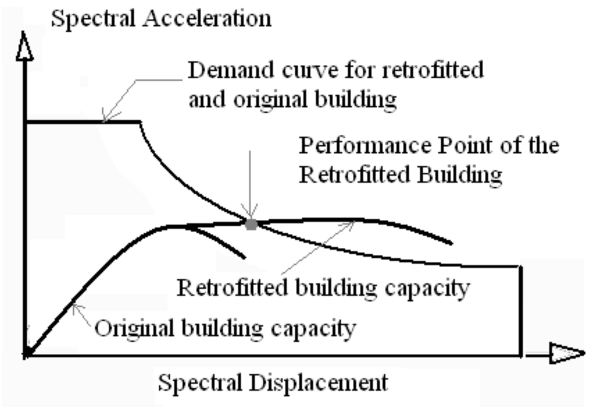


Figure 3-19. Enhancing the deformation capacity of the building

4. METAL BASED INNOVATIVE RETROFITTING TECHNIQUES FOR MASONRY WALLS

4.1. DESCRIPTION OF THE PROPOSED RETROFFITING TECHNIQUES

It is clear that masonry building retrofitting is imperiously necessary. Also the available retrofitting techniques are not always able to achieve the performance requested. In the light of arguments presented in Conclusions of Chapter2 and in the previous paragraphs a new retrofitting technique of masonry shear walls is welcome.

Strengthening of masonry walls with fibre reinforced plastics (FRP) was studied extensively in recent years. Though they have an excellent strength, fibre reinforced plastics a re brittle, being non-dissipative. Alternative techniques based on metal solutions can be used.

Within the research program new and innovative retrofitting techniques are proposed investigated and validated experimentally and numerically. The solutions use (see Figure 4-1):

- Metallic sheathing plates (SP), steel (SSP) or aluminium (ASP);
- Steel wire meshes (SWM).

The first one consists in sheeting some steel or aluminium plates either on both sides or on one side of the masonry wall. Metallic plates are fixed either with prestressed steel ties, or by using chemical anchors. The second one is derived from the FRP technique, but it applies a steel wire mesh bonded with epoxy resin to the masonry wall. Both these techniques will be described together with the experimental program and numerical simulations carried out at the "Politehnica" University of Timisoara on the aim to validate them.

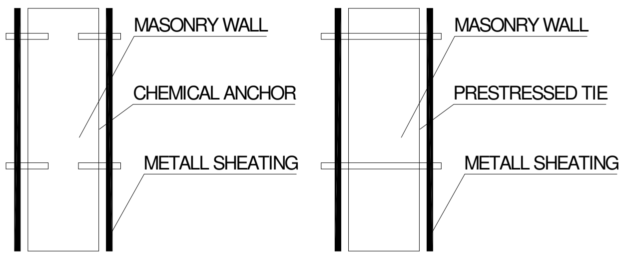


Figure 4-1. Proposed solution

The connection of the metal sheathing plates to the masonry wall is achieved in two ways: trough chemical anchors (CA) or prestressed ties (PT), placed at 200-250 mm. The spacing between connectors is imposed by the fact that is desirable to realize the connection in blocks avoiding mortar layer, being considered that in this way a better behaviour will be achieved. The wire mesh is glued using epoxy resin. Both systems can be applied on one side or both sides of the panel. It is expected that the system with metallic elements on both sides should perform better, but it isn't always possible due to architectural or functional reasons.

The suggested system provides added strength to the masonry unit for both in-plane and out-of-plane failure mechanisms. In the frame of the research the in plane behaviour will be analyzed considering that the out of plane benefit influence is obvious.

It is expected that prestressed ties to prove better performance due to more confining effect but use of chemical anchors connection can be appropriate for thick walls or when it is possible to fix the metallic plate on one side only, due to architectural reasons.

The main advantage of metallic plates in comparison with fibre reinforced plastics is their ductility, in addition to strength. Also spacing of connectors is to be determined so as to preclude elastic buckling of metallic shear panels. It is believed that the desired failure mode of the strengthening system (metallic plates and connectors) should be yielding of the metallic plate and/or plastic deformations by bearing of connectors.

Metallic plates should possess a comparative stiffness with the masonry wall, if the metallic plates are designed to dissipate seismic energy through inelastic deformations. Due to large in-plane stiffness of masonry walls, the suggested system will most probably not eliminate completely damage to masonry. A limited amount of damage to masonry has to be allowed for. Aluminium is believed to be especially suited in this case reaching "yielding stress" at smaller strain, due to a more advantageous strength to stiffness ratio than steel.

A possible strengthening of masonry walls with metallic plates or with steel wire mesh is shown in Figure 4-2. The solution can be applied on façade either on piers or spandrel. In most of the cases old masonry buildings shown weaker piers than spandrel, so to obtain a maximum benefit effect of reinforcing technique this areas are to be retrofitted.

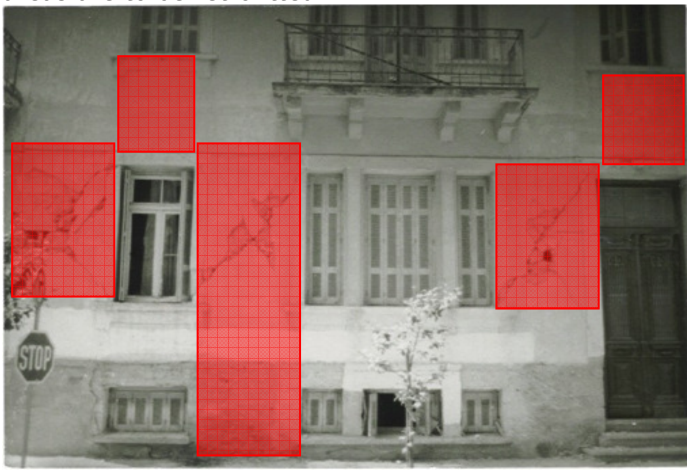


Figure 4-2. Typical damage to a masonry building and possible application of steel/aluminium plates or steel wire mesh for its rehabilitation

About reversibility of the techniques, metallic plates and steel ties can be easily removed from the masonry wall and replaced. Chemical anchors may be more difficult to remove. The most intrusive effect of the techniques is the holes in the wall. In the case of ties, local repair, such as grouting with mortar of holes will be needed if the system is to be removed completely. Wire mesh fixed with epoxy resin can be removed from the masonry wall by heating.

Metallic plates or wire mesh can be hidden if plastering is applied preserving the esthetical appearance. When masonry needs to be apparent at the facade of the building, the system can be applied from one side only (at the interior).

This techniques will be applied and tested like an reinforcing system applied on the wall surface without any connection to the adjacent elements, because like have been previously shown this connection, even if offer an amazing improvement of the behaviour in most of the cases is difficult or impossible to be done. It is expected that the system to behave better if a connection to an adjacent RC element (i.e. beam or column, or in case of infilled frames) exist. The chosen application modality will shown the minimum expected improvement.

Regarding to the benefit effect of the retrofitting techniques, due to the inserting of metallic sheathing is expected to improve shear behaviour to eliminate the shear sliding (excepting the mortar joints at the top and bottom of the wall) and maybe to change the failure mechanism from diagonal shear to rocking as a more desired failure mode. Anyway if a combined failure mechanism will be observed, this can be only in our advantage being engaging the entire capacity of the wall and for sure avoiding a brittle failure mode.

Considering this sheathing plates technique as a passive techniques is clearly that some damage must be accepted until the systems to activate and as have been already stated is expected to work as a “collapse prevention” techniques. In case of steel wire mesh it is expected to improve all the performance levels.

4.2. TECHNOLOGICAL ASPECTS

The application technology is rather simple. Metallic plates must be previously drilled. Afterwards the plate is placed on the wall, anchor holes are drilled in the masonry wall through the plate holes. The dust is blown away from the holes, followed by the injection of epoxy resin and fixing of chemical anchors (see Figure 4-3). Prestressed ties are applied similarly, but no resin is used, and the ties are tightened using a torque control wrench.

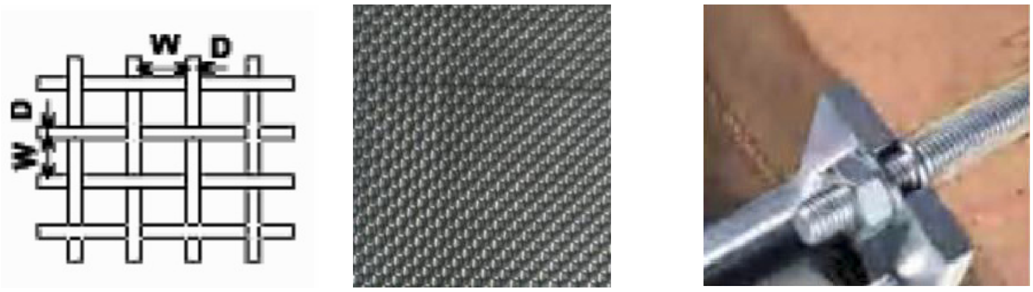


Figure 4-3. Wire mesh geometry and texture and chemical anchor

The mesh (see Figure 4-3) is produced either as galvanised steel or stainless steel bidirectional fabric. The spacing of the mesh is between 0.05 and 16 mm, while the wire diameter is between 0.03 and 3.0 mm. Tensile strength reaches 650-700 N/mm^2, while the elongation is about 45-55% in the case of stainless steel wires. For galvanised steel wire, tensile strength is usually in the range of 400-515 N/mm^2.

(a) Surface polish

(b) Resin preparation

(c) First support layer of epoxy resin

(d) Resin application

(e) SWM application

(f) Resin spreading and SWM press

Figure 4-4. Steel wire mesh (SWM) application

The application of wire mesh (see Figure 4-4) requires a previous preparation of the walls to obtain a smooth surface. The preparation of resin is similar to the one used for Fiber Reinforced Polymers (FRP). The resin is applied in two steps: a fluid layer is applied first, and after it is dried, a second thick fluid layer is applied to embed the mesh. For large surfaces the mesh should be fixed to the wall with nails in order to keep its surface plain. It is important to mention that, by heating the resin layer, the wire mesh can be removed.

All the presented techniques use mechanical connections or epoxy resin, which can be unglued by heating and can be considered reversible.

4.3. SUMMARY OF THE EXPERIMENTAL PROGRAM

In order to validate the two solutions, an experimental program was carried out at CEMSIG Laboratory [4] (director Prof. Dan Dubina) and CESMAST Laboratory [5] (director Prof. Valeriu Stoian). It included:

- Material tests (see Table 4-1);
- Preliminary tests on 500 x 500 mm specimens; (see Table 4-2);
- Full scale tests on 1500 x 1500 mm specimens, both under monotonic and cyclic loading (see Table 4-3).

Table 4-1. Material tests

Masonry component	Elastic modulus of masonry	6
	Elastic modulus of mortar	22
	Compression test on brick	6
	Compression test on mortar	88
	Tensile test on mortar	88
Steel wire mesh	Tensile test on wire	18
	Tensile test on mesh	18
Connectors	Tensile test on ties	5
Tensile test on steel plates 2mm and 3 mm		15
Tensile test on aluminium plates 5mm		5

Table 4-2. Small specimens

Preliminary	Masonry panel		3
Connection	Chemical anchor (CA)	ø8	3
		ø10	3
	Prestressed ties (PT)	ø10 – 0%	3
		ø10 – 100%	3
Diagonal tensile test	Steel wire mesh (SWM)		15
	Steel shear panel (SSP)	Chemical anchor	6
		Prestressed ties	6

Table 4-3. Large specimens

Monotonic	Reference masonry wall test		*REF*	*1*
	Steel shear panel	Chemical anchor	*SSP-CA*	*2*
		Prestressed ties	*SSP-PT*	*2*
	Aluminium shear panel	Chemical anchor	*ASP-CA*	*2*
		Prestressed ties	*ASP-PT*	*2*
	Steel wire mesh		*SWM*	*2*
Cyclic	Reference masonry wall test		*REF-c*	*1*
	Steel shear panel	Chemical anchor	*SSP-CA-c*	*2*
		Prestressed ties	*SSP-PT-c*	*2*
	Aluminium shear panel	Chemical anchor	*ASP-CA-c*	*2*
		Prestressed ties	*ASP-PT-c*	*2*
	Steel wire mesh		*SWM-c*	*2*

4.4. CALIBRATION OF THE EXPERIMENTAL MODELS

4.4.1. Summary of material tests

They are performed in order to find out the mechanical characteristics of each material, that is strength and stiffness, base material component (masonry panel) or system material (sheeting system).

In order to determine the mechanical characteristics of the sheeting system, tensile test have been performed (see Figure 4-5) on wire, mesh steel, steel and aluminium plates (and steel ties (gr. 6.8).

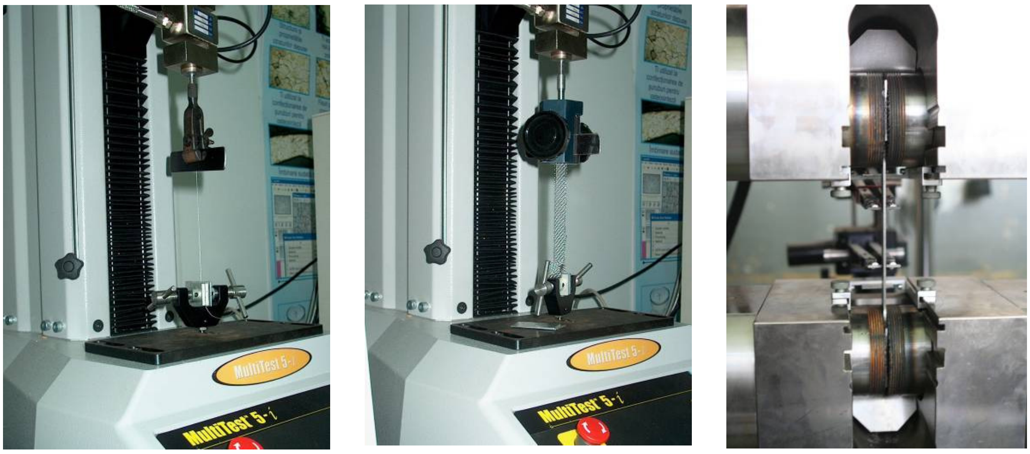
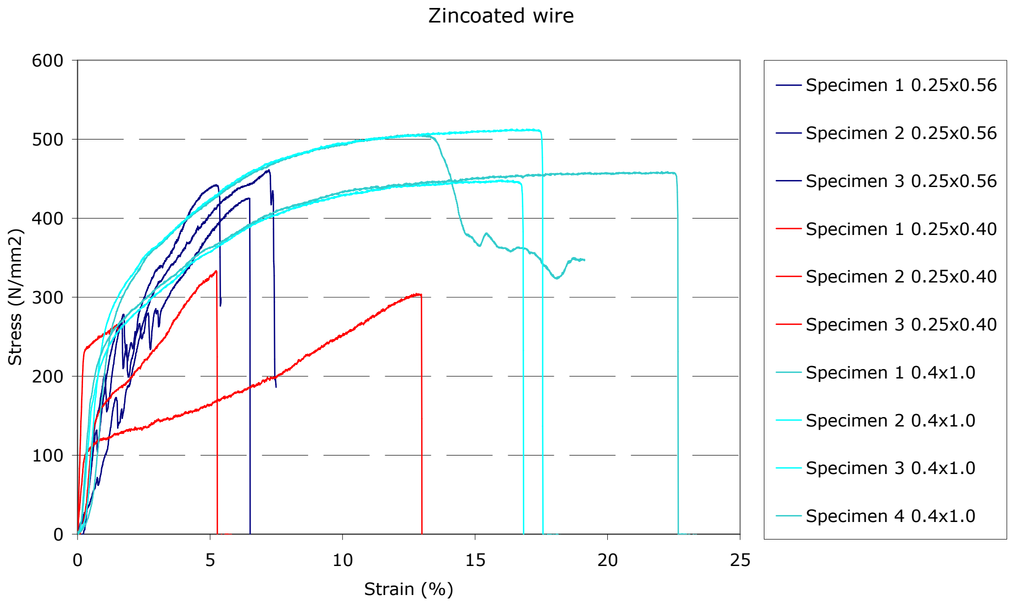


Figure 4-5. Tensile test machines (a. wires and meshes – MultiTest 5-i; b. plates and ties - UTS)

Six types of steel wire mesh were chosen: three zinc-coated (ZC) 0.25x0.40, 0.25x0.56, 0.4x1.0 and three made of stainless steel (SS) 0.3x1.25, 0.4x0.5, 0.4x1.0. The behaviour curves for wires, stainless steel and zinc-coated, is plotted in Figure 4-6 and Figure 4-7.



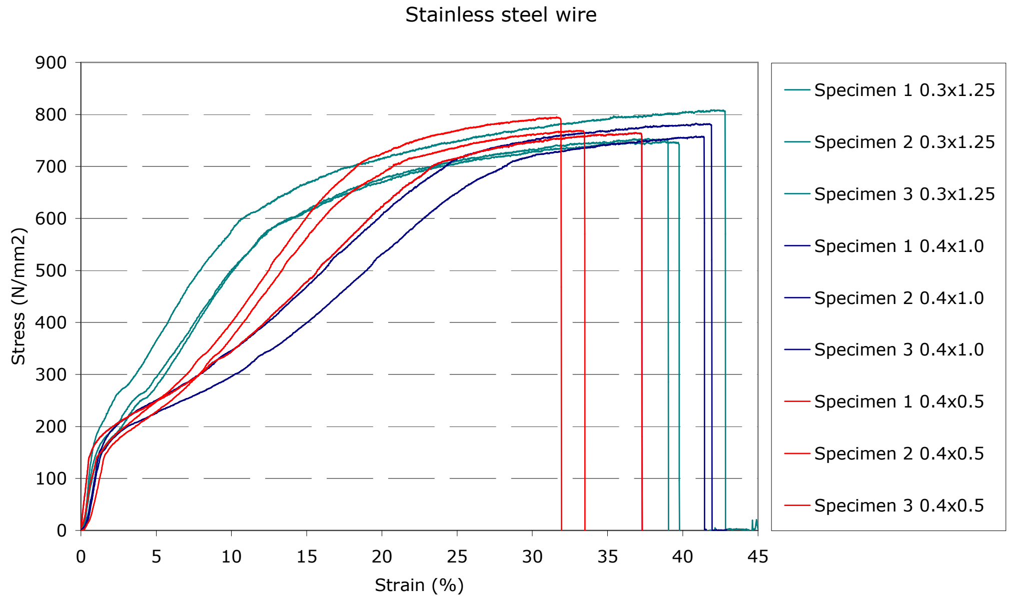


Figure 4-6. Tests results on wire a) zinc-coated and b) stainless steel

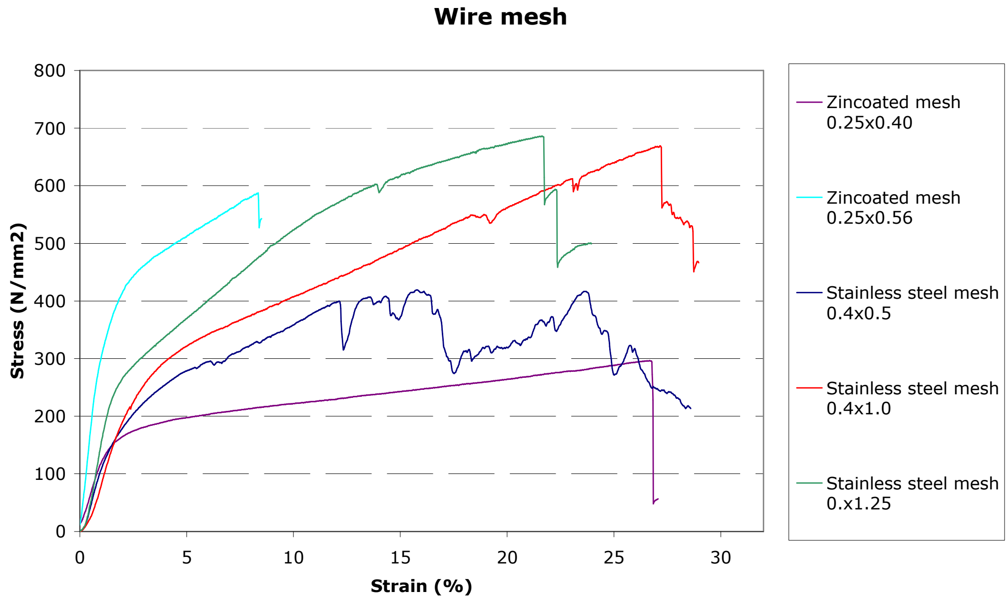


Figure 4-7. Tests results on wire meshes

In order to establish masonry characteristics, there have been performed tests on composite material (masonry) and on each component (mortar and brick) as follow: brick strength, brick elasticity modulus, mortar elasticity modulus, tension and compression strength of mortar (see Figure 4-8).

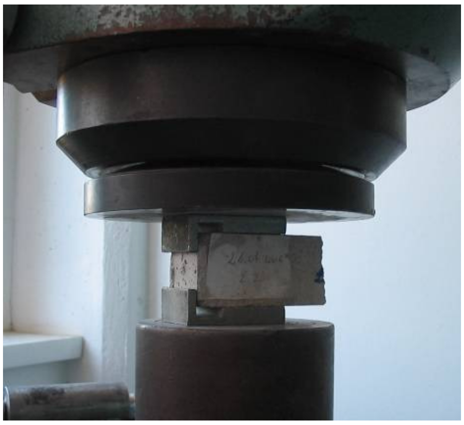
a) Compression brick

b) Elastic modulus masonry

c) Elastic modulus mortar

Figure 4-8-1. Tests on masonry and components (mortar and brick)

d) Tension mortar



e) Compression mortar

Figure 4-8-1. Tests on masonry and components (mortar and brick)

4.4.2. Analytical calibration

Some simple numerical calculations have been performed in order to determine the thickness of steel shear plates so as to obtain a rational behaviour. On this purpose three preliminary design criteria have been used expressed in terms of stiffness, local buckling and strength.

The first criterion is used in order to obtain a comparable stiffness of the metallic sheeting plates with masonry panel, so as to provide a uniform distribution of stresses between wall and sheeting. In order to evaluate the rigidity of the wall and sheeting plate, the following formulas have been used [71]:

$$k_m = \frac{1}{\frac{h_{eff}^3}{E_m I_g} + \frac{h_{eff}}{A_v G_m}} \tag{8}$$

Where k_m = stiffness of masonry panel; h_{eff} = effective wall height; E_m = longitudinal elastic modulus of masonry; I_g = moment of inertia; A_v = shear area; and G_m = transversal elastic modulus of masonry;

$$k_{plate} = \frac{1}{\frac{h_{eff}}{A_v G_s}} \tag{9}$$

Where k_{plate} = stiffness of steel plate; h_{eff} = height of plate; A_v = shear area, and G_s = transversal elastic modulus of steel [23].

By considering all material parameters known and by equating the two relations, a 2.16 mm thickness demand for the steel sheeting was obtained.

The second condition must obtain a compact plate in order to prevent local buckling and assure the dissipation of energy through plastic bearing work in connecting points only.

In order to establish the "non-compact" behaviour domain [23], the following criterion was used:

$$1.10\sqrt{\frac{K_v H}{F_{yw}}} \geq \frac{h}{t_w} \geq 1.37\sqrt{\frac{K_v H}{F_{yw}}} \tag{10}$$

Where K_v = plate buckling coefficient; H= horizontal load of the panel; F_{yw} = yielding stress of steel; h = distance between connectors (imposed by masonry texture); and t_w = steel plate thickness.

From equation, the compactness criterion results as t_w>2.27 mm.

A more complex methodology can be used in order to evaluate the resistance of each component of the system, proposed by the producer of chemical anchor [95]. Three components govern the behaviour of chemical connection, e.g. the matrix (masonry with epoxy resin), steel anchor and steel plates. It is believed that the most desirable failure mode is the bearing of the steel hole (e.g. in the connecting points). In order to obtain this failure mode, the bearing resistance should be less than the minimum between the shear resistance of connector and the crushing resistance of the matrix.

$$N_{bearing} \leq min(N_{masonry}, N_{conector}) \tag{11}$$

For chemical anchors, the design methodology suggested by producer (Hilti-Catalogue, 2005) has been adapted for the masonry matrix e.g.

$$V_{Rd,c} = V^{0}_{Rd,c} \cdot f_{BV} \cdot f_{\beta V} \cdot f_{AR,V} \tag{12}$$

Where VRd,c = matrix edge resistance; $V_{Rd,c}{}^{0}$ = basic matrix edge resistance; f_{BV} = matrix strength influence; $f_{\beta V}$ = load direction influence; and $f_{AR,V}$ = spacing and edge coefficient.

Two cases were considered: ø8 and ø10 for the connector diameter. Corresponding plate thickness amounted to 2.20 and 2.48 mm.

It was decided to use a 3 mm thickness steel plate of S235 grade when applied on one side and 2 mm thickness plate of S235 grade when applied on both sides. Alternatively, 5mm aluminium plates have used (99.5% Al 1050 H14 ($R_{p0.2\%}$=105 N/mm2)).

4.5. CALIBRATION TESTS

It was decided to perform a series of experimental tests on small specimens in order to validate and calibrate the proposed techniques carried out in the CESMAST Laboratory of Department of Civil Engineering from the "Politehnica" University of Timisoara. The tests on small specimens were carried out for the study of the connection behaviour and strengthening solution calibration.

The small experimental specimens were 50 cm wide, 50 cm high and 25 cm tick, built of solid clay bricks with dimensions 6.3 x 24.0 x 11.5 cm and unit strength 9.0÷10.0 N/mm^2 and cement based mortar (cement : sand ratio was 1:1) with strength 30÷50 N/mm^2.

The preliminary tests (diagonal tension - shear test) were carried out on unreinforced masonry panels to obtain reference values for the virgin specimen.

4.5.1. Connection tests

Connection push-tests were performed in order to choose the connector diameter and to assess the influence of prestress level of steel ties. The experimental set-up is presented in Figure 4-9.

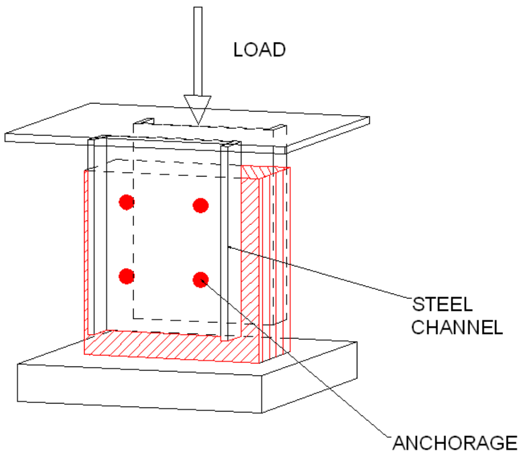


Figure 4-9. Experimental set-up and testing machine for connectors

Connection tests were performed in order to establish the connector diameter and to assess the influence of prestress level of steel ties. The aim was to harmonize the crushing resistance of the matrix (masonry + resin) and shear of the steel connector (for possible failure modes see Figure 4-10) and to assure as much as possible the integrity of the masonry element. The testing device was made of two back-to-back cold formed channel profiles (f_y = 350 N/mm^2) with 3mm thickness of the walls. The spacing between connectors was 200 x 225 mm, imposed by the masonry texture.

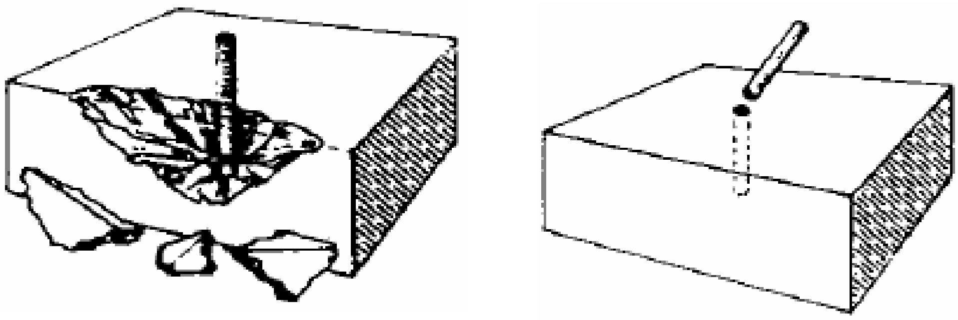


Figure 4-10. Shear failure modes of chemical anchors [95]

4.5.1.1. Chemical anchors

Chemical anchors ø8 and ø10 diameters gr.5.8 have been tested. The failure mode for ø8 was the shear of connector and for ø10 the shear of connector and crushing of masonry (see Figure 4-11). The chemical anchors are produced by Hilti, and the commercial name is HIT HY 50.

For the large specimen tests, an ø10 connector was chosen, due to the more efficient behaviour and resistance (see Table 4-4).

Table 4-4. Chemical anchor connection

	F (ton)	d (mm)	$F_{conector}$
CA ϕ8-1	10.1	8.02	
CA ϕ8-2	8.8	12.5	1.15
CA ϕ8-3	8.5	7.87	

ø 8 chemical anchors

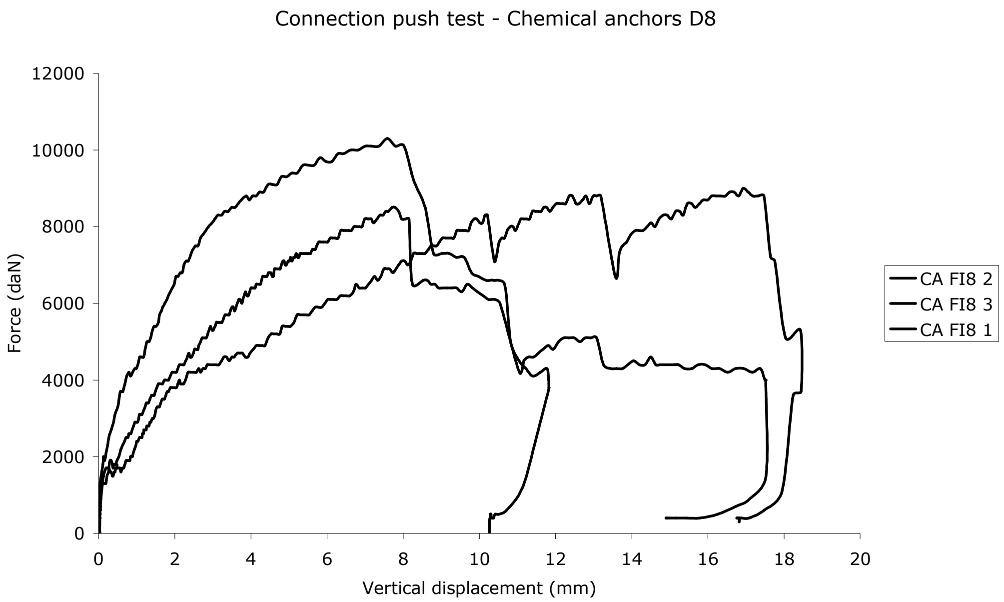
	F (ton)	d (mm)	$F_{conector}$
CA ϕ10-1	10.1	11.37	
CA ϕ10-2	9.3	14.07	1.35
CA ϕ10-3	12.6	18.02	

ø 10 chemical anchors

a) ø8 diameter

b) ø10 diameter

Figure 4-11. Failure modes for chemical anchor connections



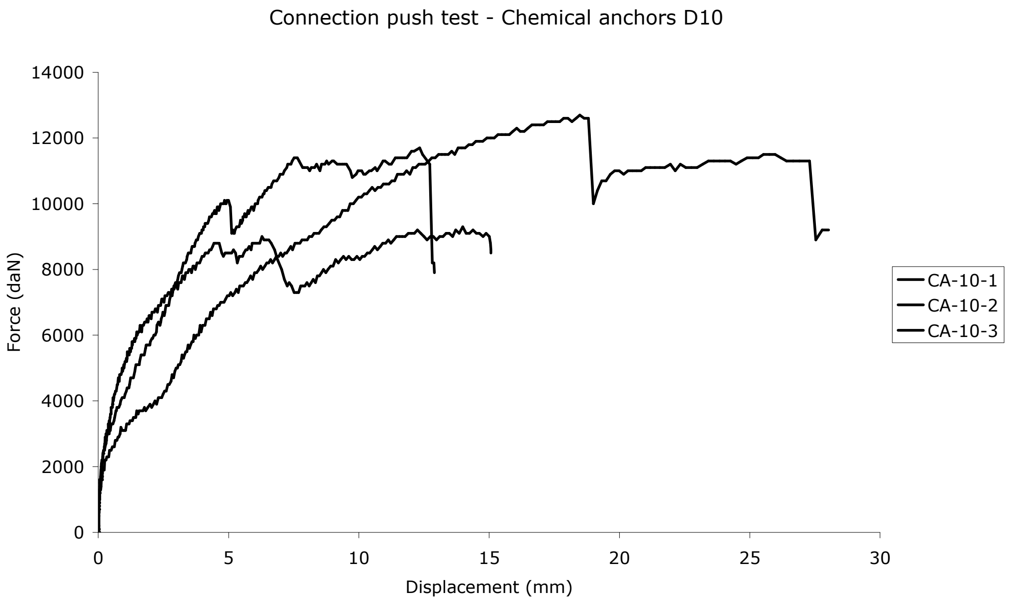


Figure 4-12. Behaviour curves for ø8 a) diameter and ø10 b) chemical anchors

4.5.1.2. Prestressed ties

Starting from the experience with chemical anchors an ø10 ties were chosen. Two prestressing levels have been applied for the ø10 ties gr.5.8 (i.e. snug tightened ties (0% prestress) and full prestress (100%) – M_t = 35Nm). The failure mode was shear on ties, masonry specimens remaining almost intact (see Figure 4-13).

Figure 4-13. Behaviour of prestressed ties connections

It was noted that the prestress level has increased the resistance of connections due to the confinement of masonry. In comparison with chemical anchors, the connection with prestressed ties is more resistant and more rigid.

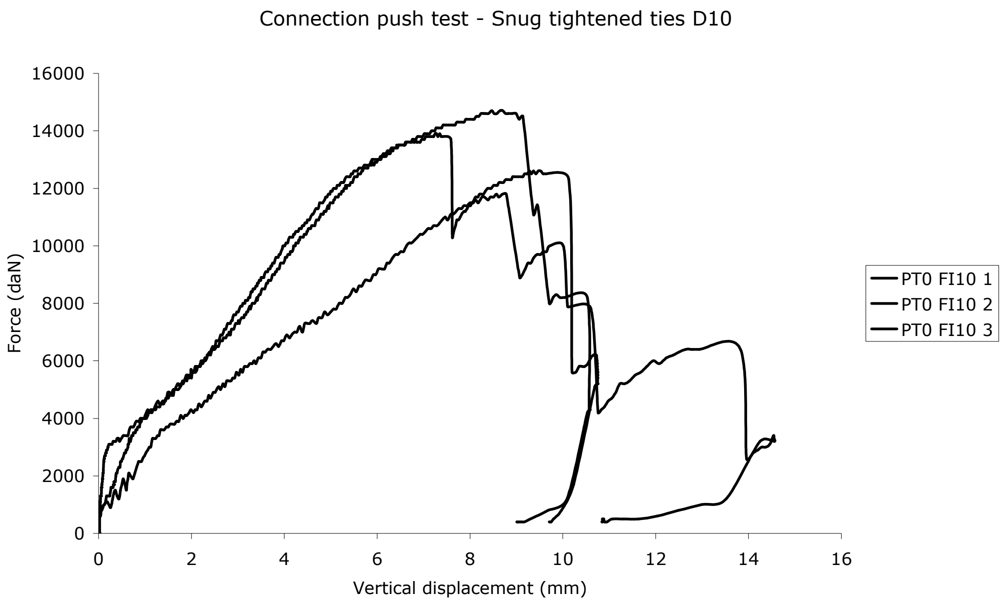
Table 4-5. Prestressed tie connection

	F (ton)	d (mm)	$F_{conector}$
PT0-1	11.8	8.78	
PT0-2	13.9	7.35	1.68
PT0-3	14.7	8.7	

Snug tightened ties (0%)

	F (ton)	d (mm)	$F_{conector}$
PT1-1	14.8	6.92	
PT1-2	13.6	10.0	1.75
PT1-3	13.7	8.46	

Full prestressed ties (100%)



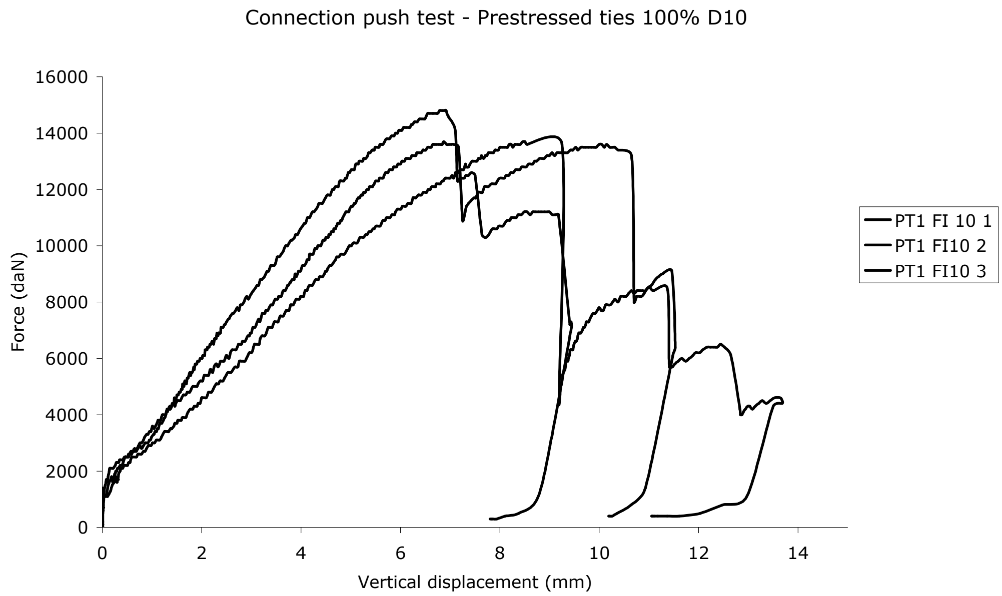


Figure 4-14. Behaviour curves for ø8 a) diameter and ø10 b) Prestressed ties

4.5.1.3. Connection test results

A more resistant and rigid behaviour of the prestressed ties was observed in the case of connections, in comparison with chemical anchors. The level of prestress have also improved the behaviour of the connection. As one can observe from the failure mode, an important advantage of the ties is that the load is distributed on the entire thickness of the wall without the introducon of a lever arm that induced negative effects. Even if little bearing of the hole was observed, we may conclude that the behavioural curves previous plotted represent the behaviour of the connection in the relation between masonry and the connector type.

4.5.2. System tests

Tests on retrofitting systems were carried out in order to validate the analytical assumption regarding of shear plates and to choose a proper steel wire mesh. The experimental set-up on small specimens and a sample test on unreinforced masonry panel are presented in the Figure 4-15. The ultimate load capacity of the unreinforced panel was 8 t.

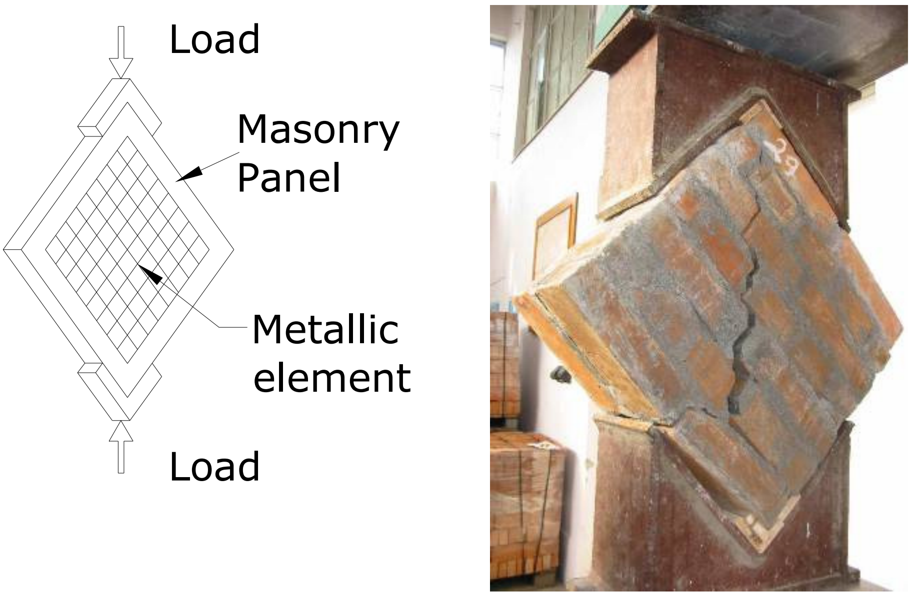


Figure 4-15. Experimental set-up for split test

The experimental set-up on small specimens and a sample test on unreinforced masonry panel are presented in the Figure 4-15.

Connection realized with chemical anchors

Connection realized with prestressed ties

Figure 4-16. Failure mode of steel SP applied on one side (SSP3)

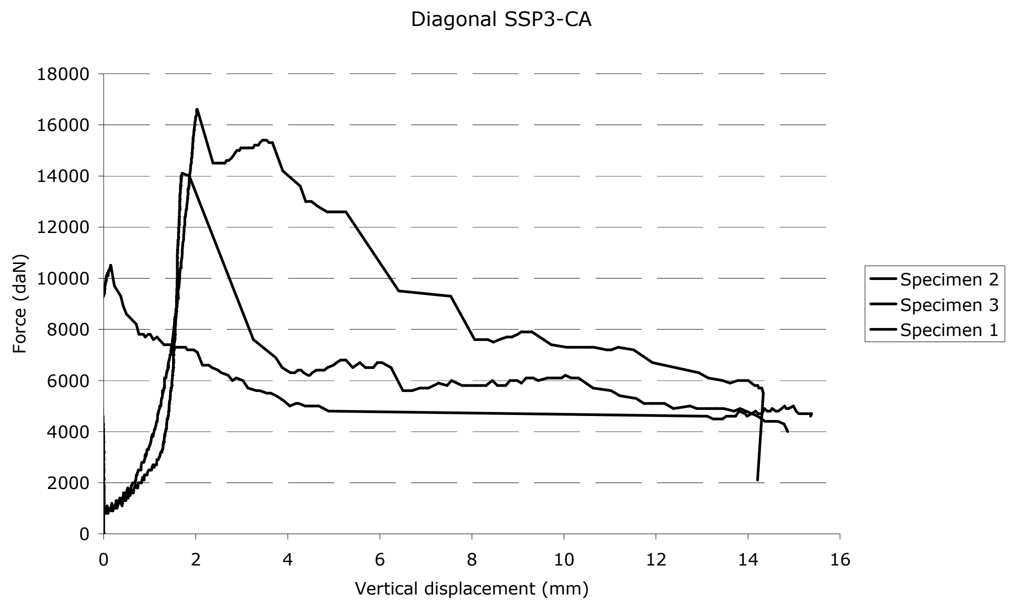
4.5.2.1. Metal sheeting plates (SP)

Steel shear plates S235 grade of 2 mm thickness on both side (see Figure 4-18) and 3 mm thickness on one side (see Figure 4-16), connected with chemical anchors (CA) and prestressed ties (PT) were tested.

The results are summarized in Table 4-6.

Table 4-6. Results for SP applied on one side (SSP3)

	Load (ton)			Vertical displacement (mm)			Horizontal displacement (mm)		
	1	2	3	1	2	3	1	2	3
CA	16.6	14.1	10.5	4.0	3.4	0.3	0	0	0.6
PT	12.4	15.2	9	2.4	1.5	0.5	0	0	0



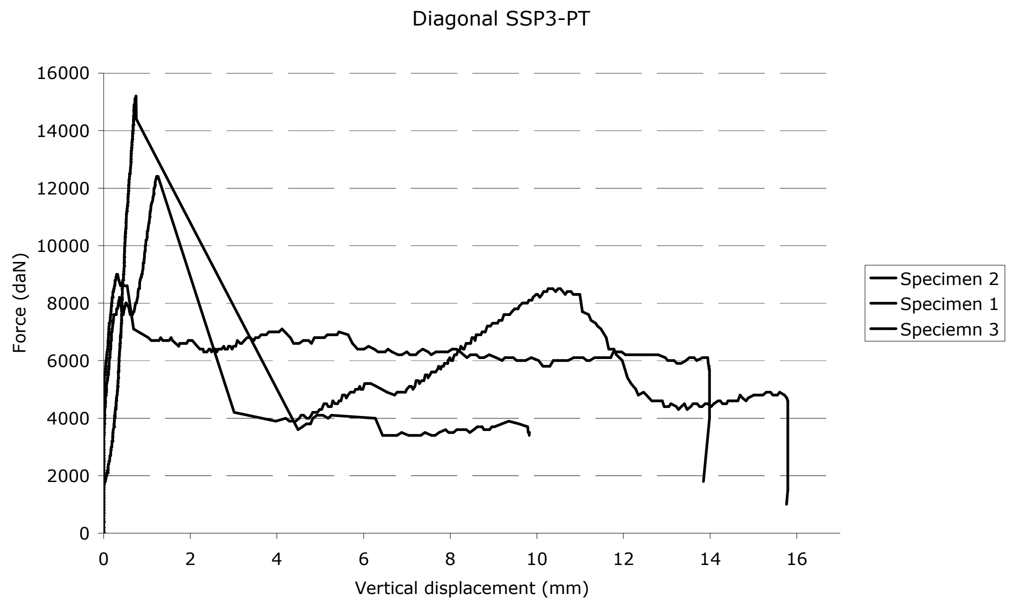
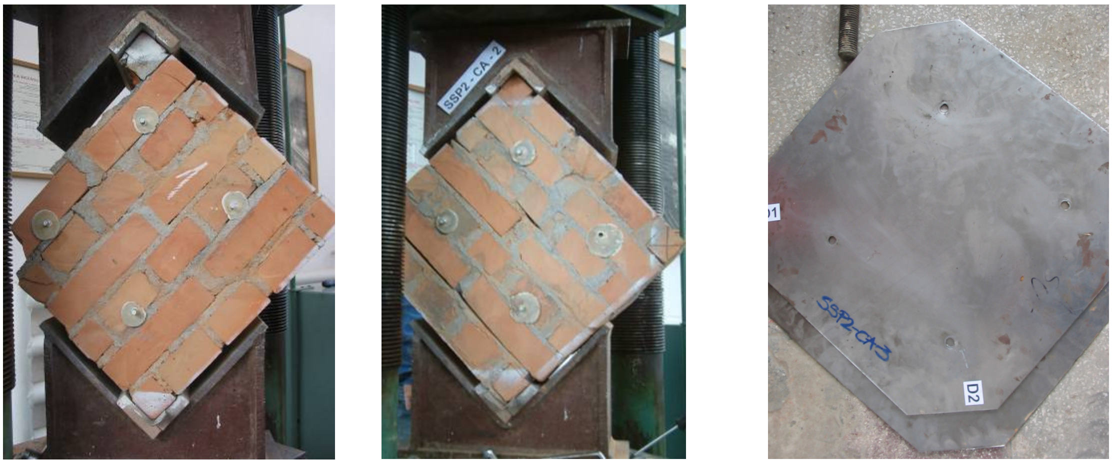


Figure 4-17. Behaviour of masonry panel sheathed with steel SP applied on one side (SSP3) and connected trough chemical anchors (CA) and prestressed ties (PT)



Connection realized with chemical anchors

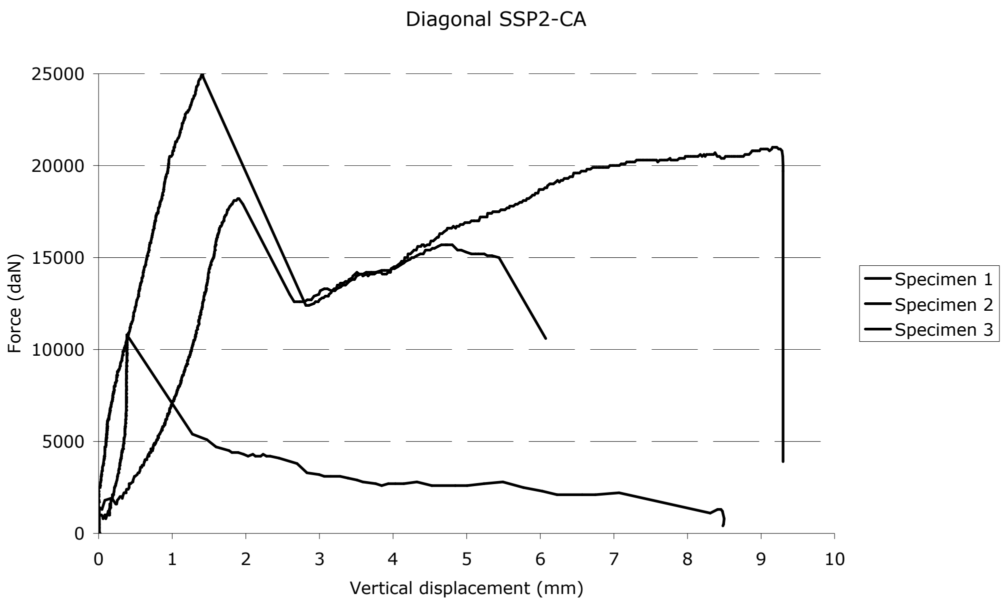
Connection realized with prestressed ties

Figure 4-18. Failure mode of steel SP applied on both sides (SSP2)

The results are summarized in Table 4-7.

Table 4-7. Result for SP applied on both sides (SSP2)

	Load (ton)			Vertical displacement (mm)			Horizontal displacement (mm)		
	1	2	3	1	2	3	1	2	3
CA	18.2	10.8	25	3.6	0.8	2.8	0.2	0	0
PT	35.9	12.5	13.3	4.8	2.5	5.4	0	0.3	0.4



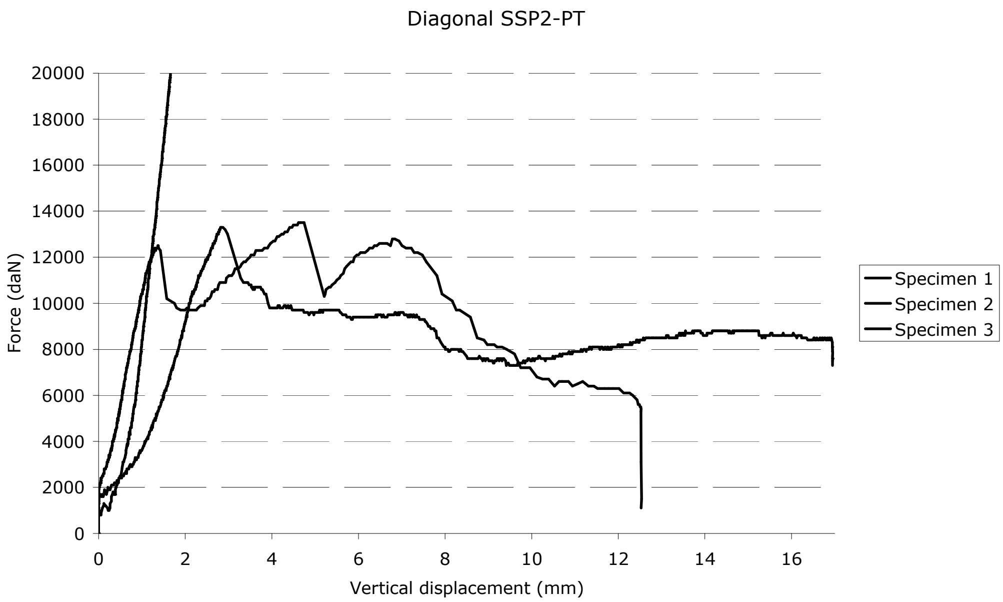


Figure 4-19. Behaviour of masonry panel sheathed with steel SP both sides (SSP2) connected through chemical anchors (CA) and prestressed ties (PT)

Figure 4-20. Bearing of steel plates applied on both sides (SSP2)

4.5.2.2. Steel wire mesh (SWM)

There was no analytical procedures for the design of steel wire mesh reinforced masonry; therefore, the calibration was based on the experimental tests. The purpose of tests was to select the appropriate resin and wire mesh to be applied on large specimens. In the first step there were chosen six types of wire mesh, zinc coated (ZC) and stainless steel (SS), to be bonded on one side. The results are summarized in Table 4-8.

Table 4-8. Results for SWM applied on one side

		Load (ton)	Vertical displacement (mm)	Horizontal displacement (mm)
M1	ZC 0.25x0.4	20.9	3.09	0.61
M2	ZC0.25x0.56	15.3	1.74	0.03
M3	ZC 0.4x1.0	8.5	N.A.	N.A.
M4	SS 0.3x1.25	14.7	3.66	0.16
M5	SS 0.4x0.5	44.70	N.A.	N.A.
M6	SS 0.4x1.0	19.3	3.83	0.83

Compare with FRP technique a less fluid resin was selected. In order not to change too many parameters and based on the experimental results, the following wire meshes were chosen: zinc coated (ZC) 0.4x1.0 (*D* x *W*), stainless steel (SS) 0.4x0.5 and 0.4x1.0 to be applied on both sides.

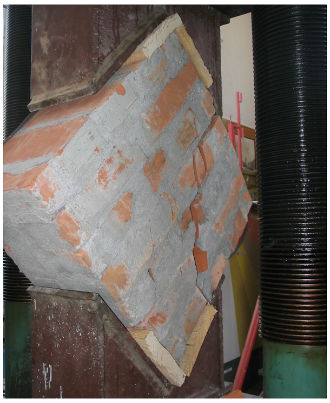
The following failure modes were observed (see Figure 4-23).


a) ER M1-1


b) ER M2-1


c) ER M3-1


d) ER M4-1


e) ER M5-1


f) ER M6-1

Figure 4-21. Failure mode for SMW applied on one side

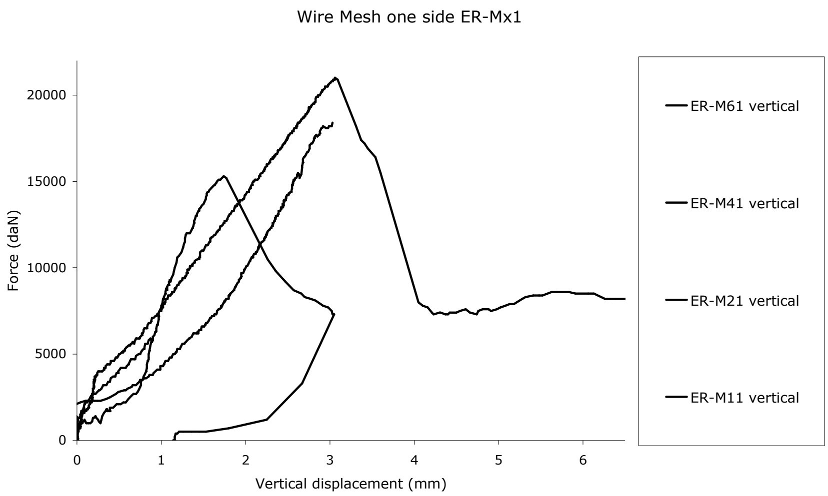
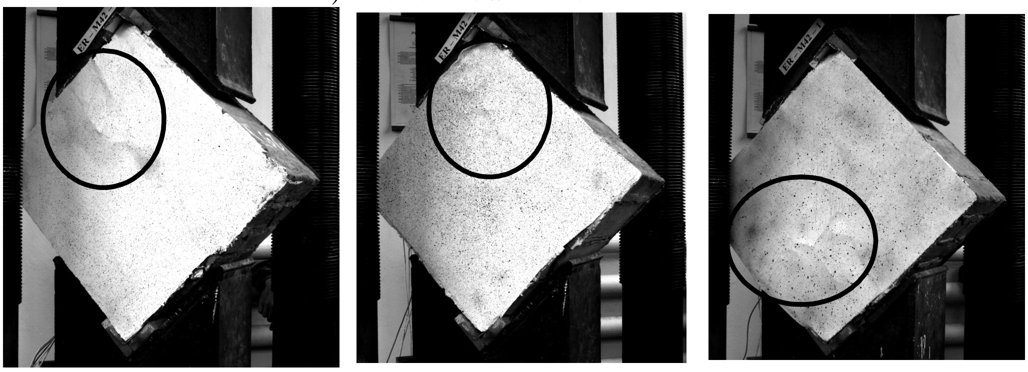
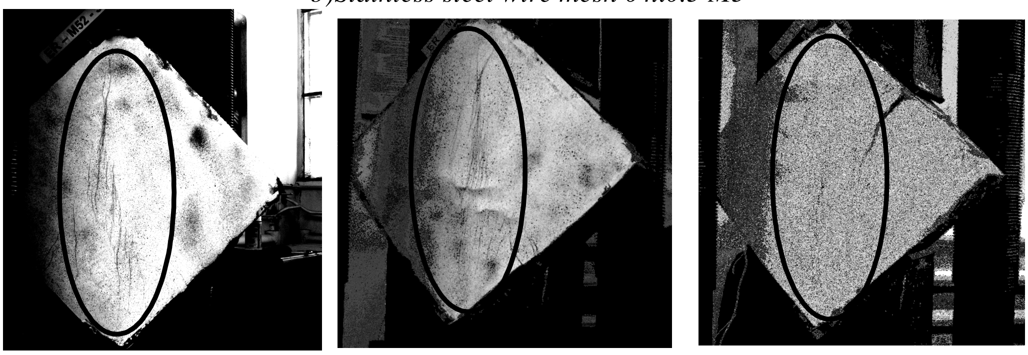


Figure 4-22. Behaviour curves for SMW applied on one side

a) Zinc coated wire mesh 0.4x1.0 M3



b)Stainless steel wire mesh 04x0.5 M5



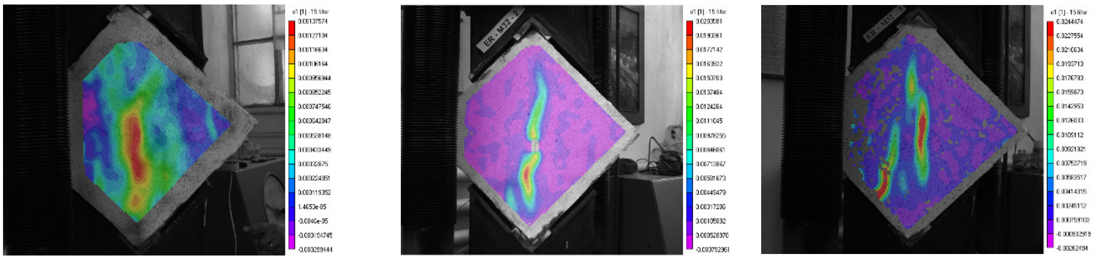
c) Stainless steel wire mesh 04x1.0 M6

Figure 4-23. Failure mode for SMW applied both sides

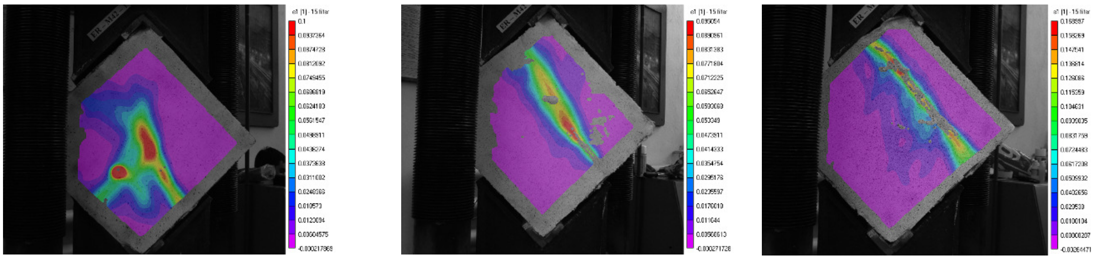
- WM3 (ZC 0.4x1.0) – sudden wire mesh rupture simultaneous with masonry crack – resistance improvement (weak WM)
- WM5 (SS 0.4x0.5) – debonding of wire mesh, rupture in resin – strength improvement, energy dissipation due to the successive debonding (strong WM)
- WM6 (SS 0.4x1.0) – wire mesh yield – improvement of resistance and ductility (optimal).

Based on these observations, the stainless steel 0.4x1.0 was chosen to be applied on large specimens.

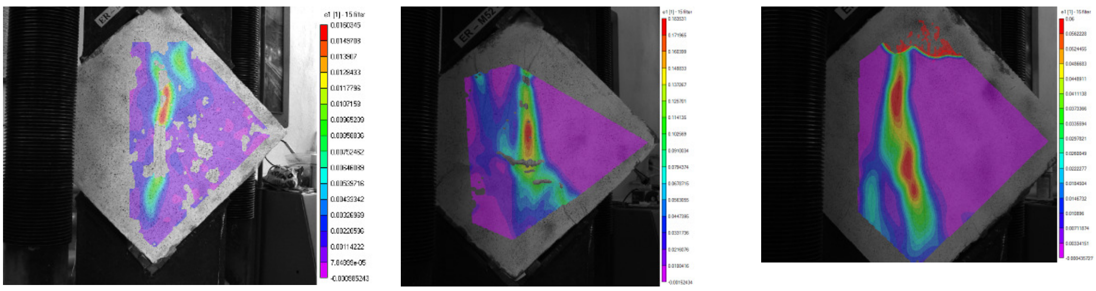
Internal deformation (principal tensile strain) at failure point for each of the specimens retrofitted with steel wire mesh on both sides, measured with Vic3D - Limess (see Figure 4-24).



Zinc coated 0.4x1.0 (1, 2 and 3)



Stainless steel 0.4x0.5 (1, 2, and 3)

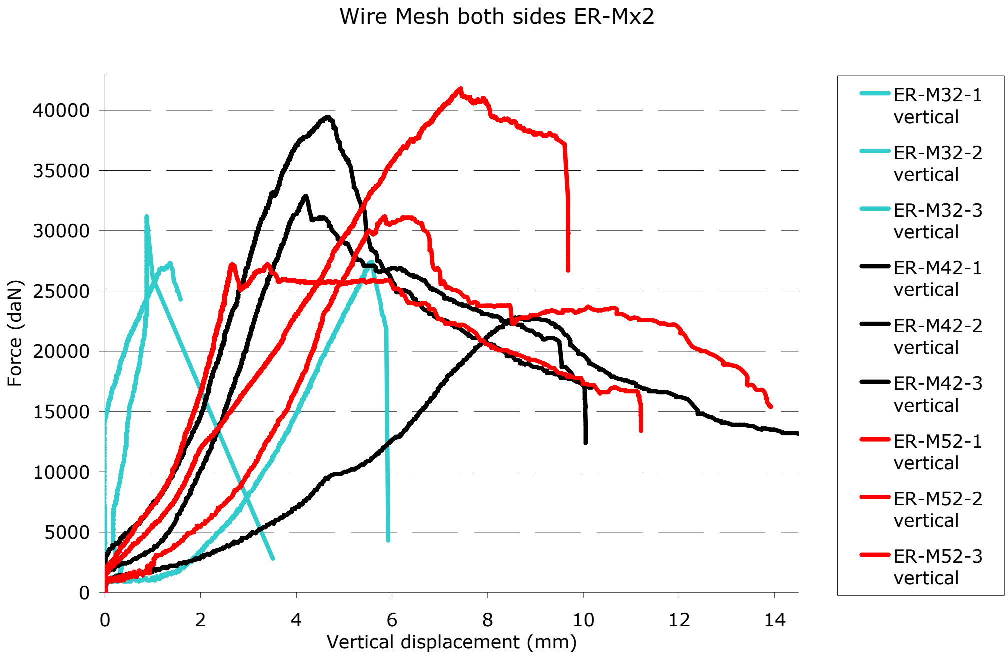


Stainless steel 0.4x1.0 (1, 2 and 3)

Figure 4-24. Failure mode for SMW applied on both sides

Table 4-9. Results of SWM applied on both sides

	Load (ton)			Vertical displacement (mm)			Horizontal displacement (mm)		
	1	2	3	1	2	3	1	2	3
WM3	31.2	27.3	27.1	0.8	5.5	1.3	0	0	0
WM5	31.1	27.1	41.1	6.2	2.7	7.6	0.7	0	0.8
WM6	31.1	22.8	39.4	4.5	8.9	4.6	0.4	0.8	0.4



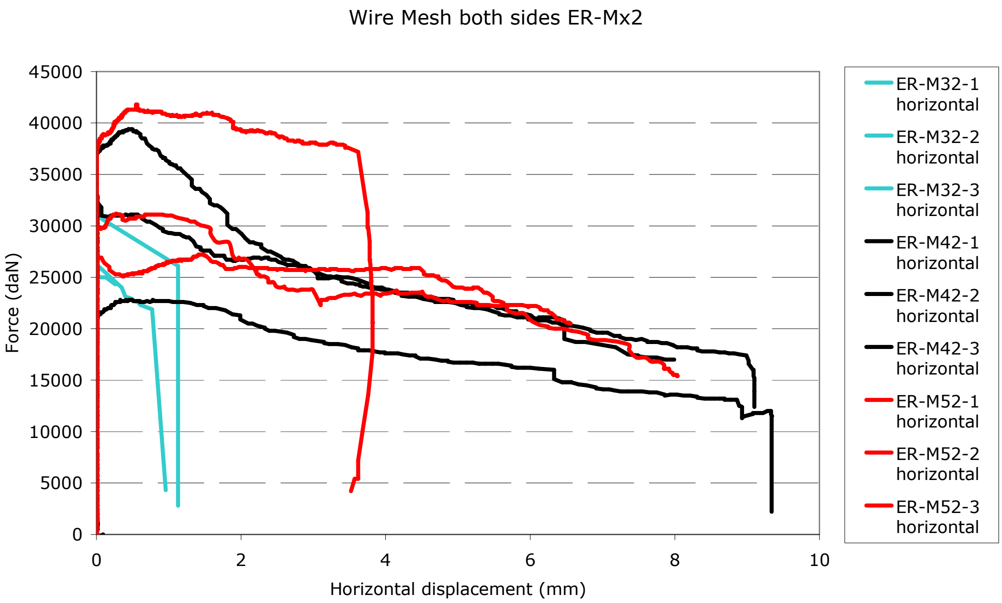


Figure 4-25. Behaviour curves for SMW applied on both sides

4.6. TESTS ON LARGE MASONRY SPECIMENS

4.6.1. Introductory remarks and testing equipments

The tests were carried out in two different experimental frames, one for monotonic loading (see Figure 4-28), in the CESMAST Laboratory of Civil Engineering Department, and one for cyclic loading (see Figure 4-31), in the CEMSIG Laboratory of Department of Steel Structure and Structural Mechanics, both from the "Politehnica" University of Timisoara.

The experimental specimens were 150 cm wide, 150 cm high and 25 cm thick, build from solid clay bricks with dimensions of 6.3x24.0x11.5 cm and unit strength 9.0÷10.0 N/mm^2 and cement based mortar with strength 6÷10 N/mm^2. The dimension of the tested wall was chosen in order to respect the test set-up capabilities and to correspond to the dimensions of a commonly used wall pier, trying not to use scaling. The retrofitting systems were applied on uncracked panel. The dimension of the steel and aluminium sheathing and the connectors spacing is show in Figure 4-26. For specimens sheathed on both sides a 2mm thickness steel (S235) plate was used, and for one side 3 mm thickness. In the case of aluminium (99.5% Al 1050 H14) sheathing, 5 mm thickness plate was used in both cases.

For the test under monotonic conditions there have been used hydraulic jacks of ~500 kN load capacity and 160 mm displacement capacity. The frequency of the load used was 18.70 kN/min, which corresponds to an approximately 0.92 mm/min.

The equipments like: transducers used for measuring the behaviour of the tested system, data acquisition system used for collecting the data are produced by Almemo - 3290-8 data logger, 5990-0 data logger, displacement tracer 100mm, displacement transducer – 150mm, pressure sensor, 600 bar, universal connectors for strain gages. There has also been used dedicated Almemo software, AMR Demo version 5.

The testing under dynamic conditions was carried out using two hydraulic actuators QUIRI with the following characteristics:

- Capacity
    - Actuator 1: 422 kN under dynamic loading; 563 kN under static loading;
    - Actuator 2: 950 kN under dynamic loading; 1267 kN under static loading;
- Stroke: +/-200 mm;
- Computer operation;
- Displacement or force control;
- Predefined, as well as user defined loading procedures;
- Built-in displacement and force transducers;

There were used linear displacement potentiometer transducers produced by Novotechnik, with enclosed return spring and stroke length of 100 and 200 mm.

For data acquisition there was used a HP3852A data logger with the following characteristics:

- 40 channels for transducers, as follows: 30 channels for resistive transducers; 4 analogous channels; 6 channels for inductive transducers;
- 50 channels for strain gauges

The cyclic load was applied in displacement control by respecting the ECCS procedure for dynamic loading.

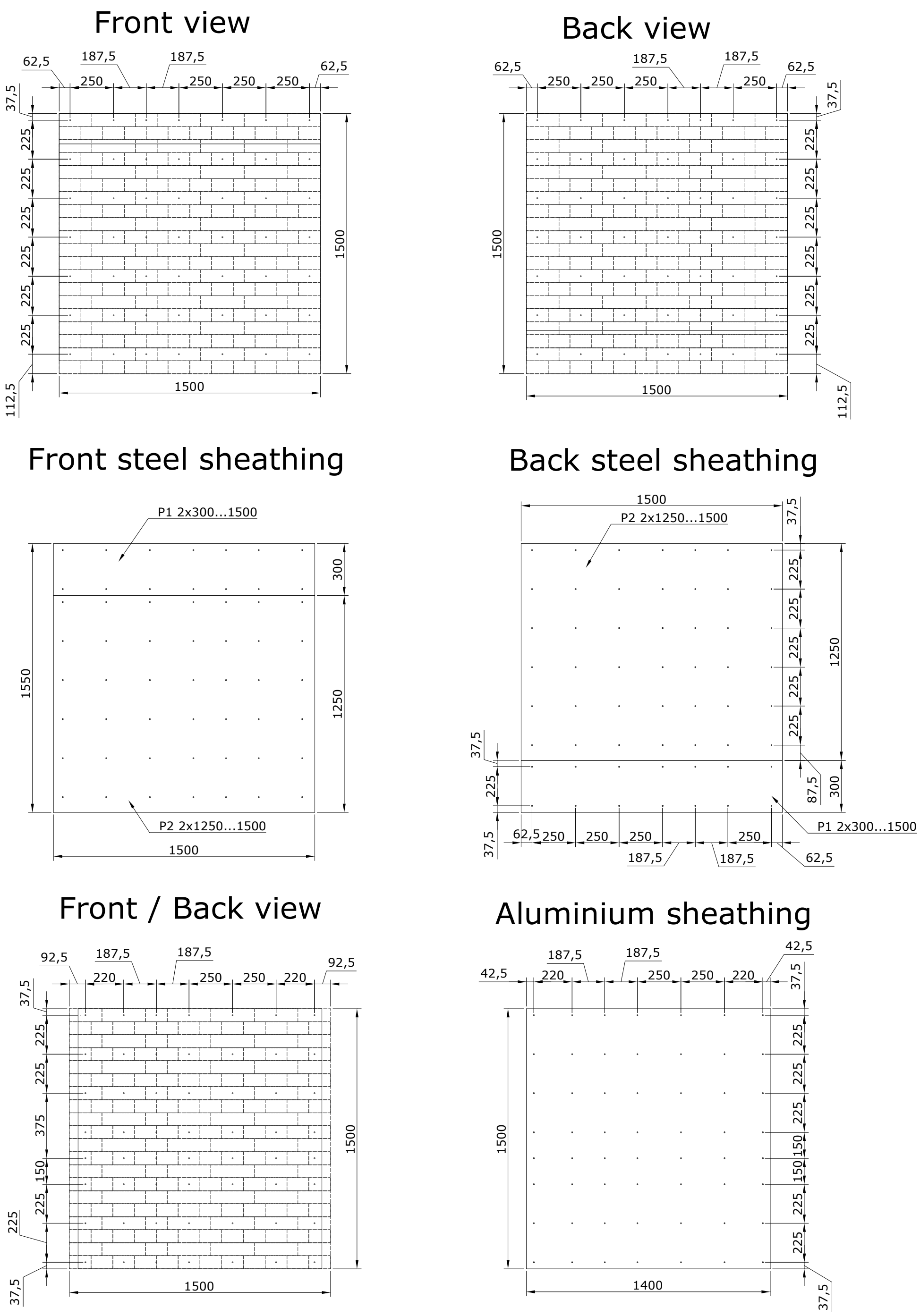


Figure 4-26. Experimental specimens

For the interpretation of experimental results the procedure showed in Figure 4-27 has been applied. For all the experimental results the envelope curves were determined and the key points were established, representing the elastic limit, maximum point and ultimate point. The elastic point was determined as the intersection of initial stiffness (generally chosen at 70% of maximum force) and a post-yielding stiffness (considered as 10% of the initial stiffness k_0) tangent to the envelope experimental curve. The ultimate point was taken at 20% loss of the maximum force. These points have been used for a more easy representation of the results in a parametric manner and also for the comparative presentation of the results. This parametric expression can also be used in the case of simple "equivalent strut" (see 5.1.2.3) for global analysis.

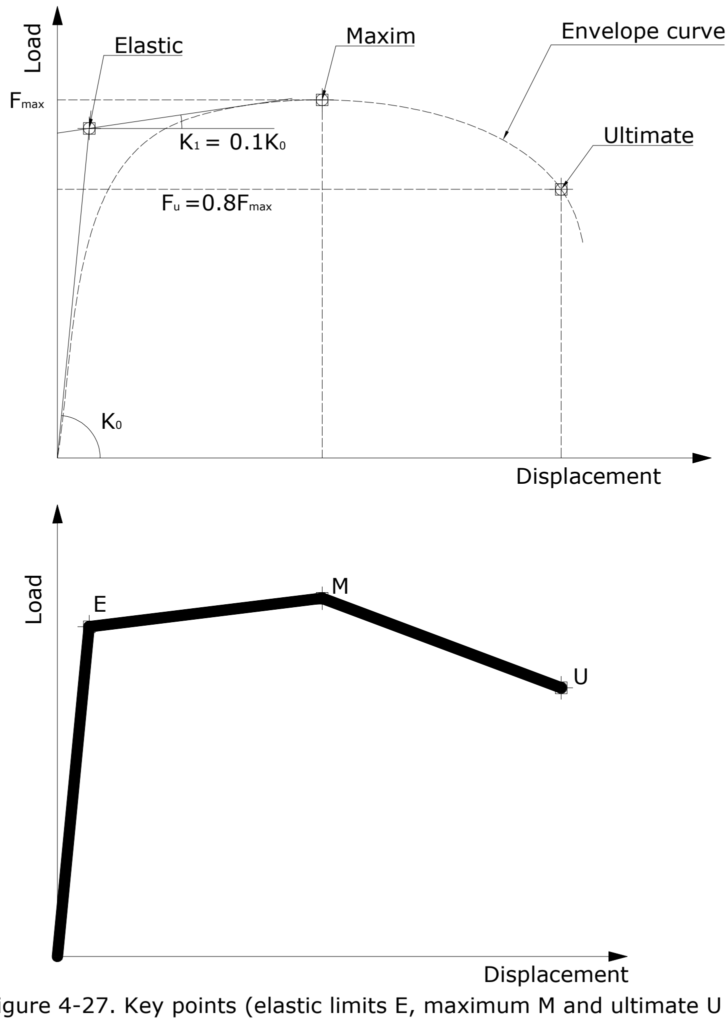


Figure 4-27. Key points (elastic limits E, maximum M and ultimate U points)

4.6.2. Testing of large specimens under monotonic conditions

For the **monotonic** loading conditions there was used a device composed of a pair of L-shaped solid steel elements attached to the massive reinforced concrete block, which was part of the wall at the top and the bottom. The top L-shape beam has a joint that blocks the vertical movement and allows element horizontal displacement. The forces have been applied by the help of hydraulic jacks. The constant vertical force was applied on the top of the specimen, acting through the reinforced concrete bond beam. The monotonic increasing horizontal force (shear) was applied through a series of steel bolts embedded in the reinforced concrete block and mounted to the L-shaped steel elements at the top as well as at the bottom (see Figure 4-28).

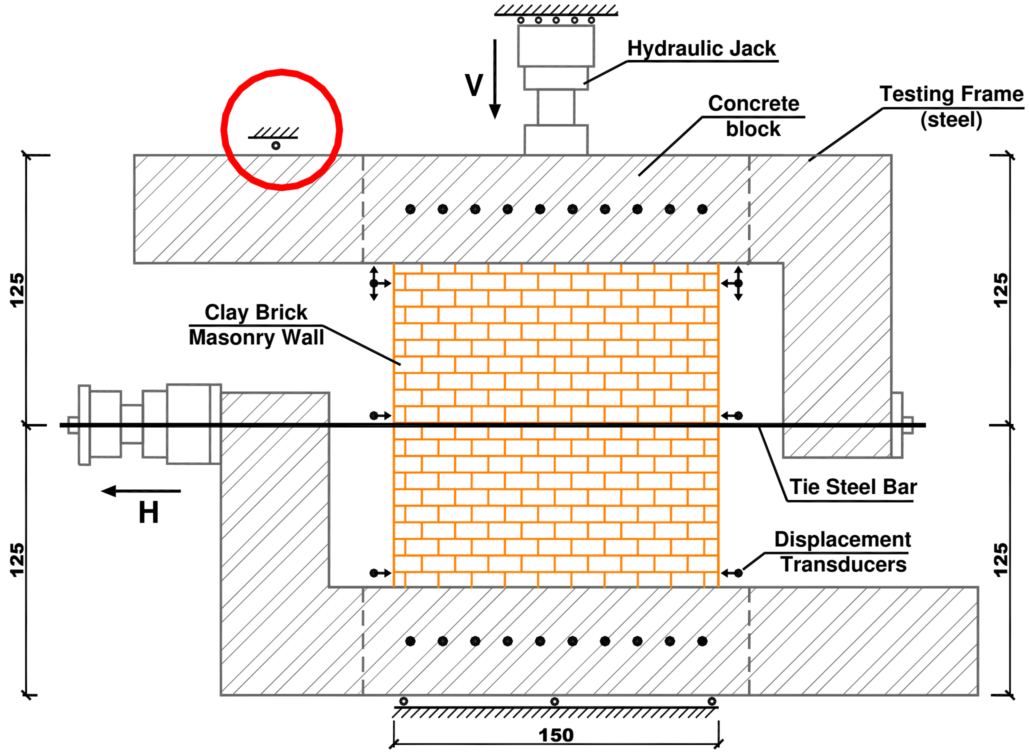


Figure 4-28. Testing frame for monotonic loading [166]

The deformations of the wall were measured by linear displacement transducers, which were placed along the height of the wall, on the left and the right sides, measuring the horizontal displacement of the specimen. Other transducers measured the vertical displacements on each side of the specimen, being placed on the steel frame at the first and the last mortar bed joints, respectively.

The specimens were retrofitted with different solutions and then tested up to failure. The recorded data were the horizontal load, the horizontal and vertical displacement, the strain (to the steel wire mesh retrofitting system) and the specimen failure modes. By considering a rigid upper beam with restrained rotations, in Figure 4-30 are plotted the results in terms of load and absolute lateral displacement (top transducer – bottom transducer recorded displacement) that include both deformation due to shear and bending of the testing specimens.

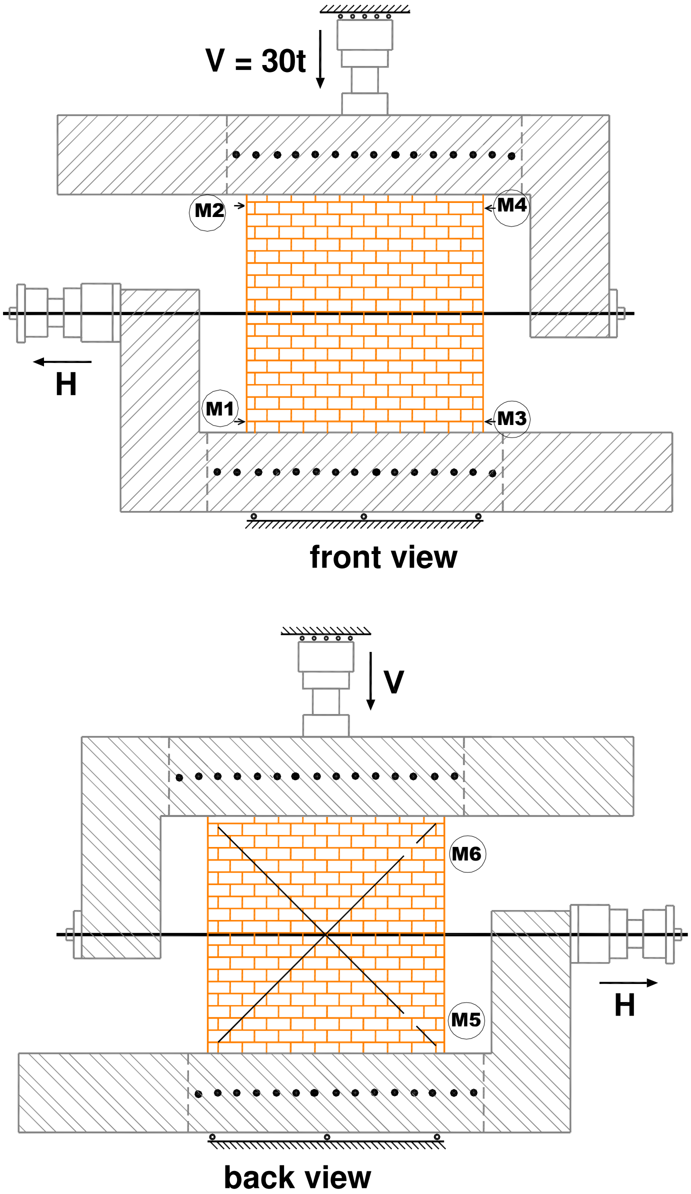
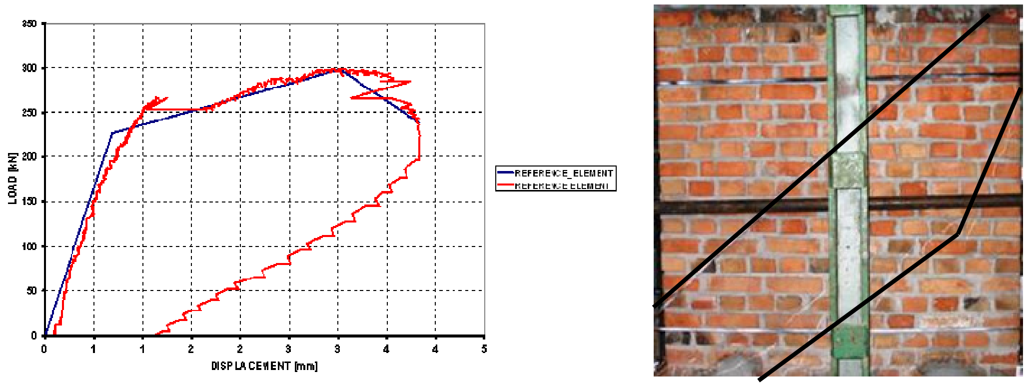
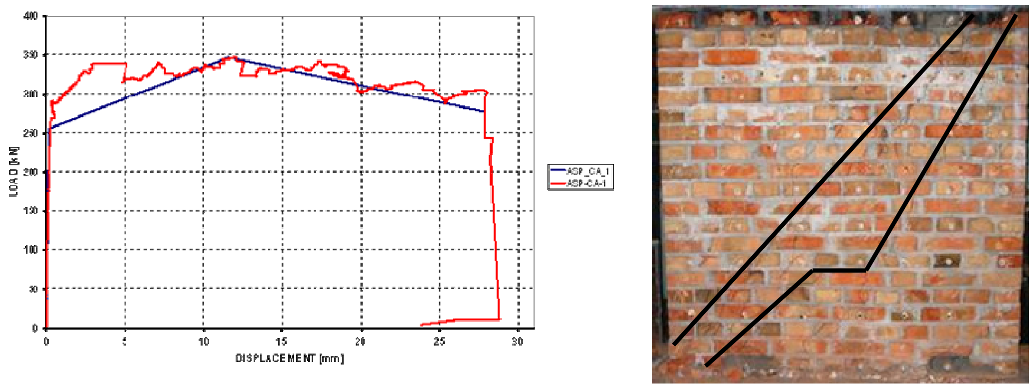


Figure 4-29. Testing principle scheme and displacement transducer position

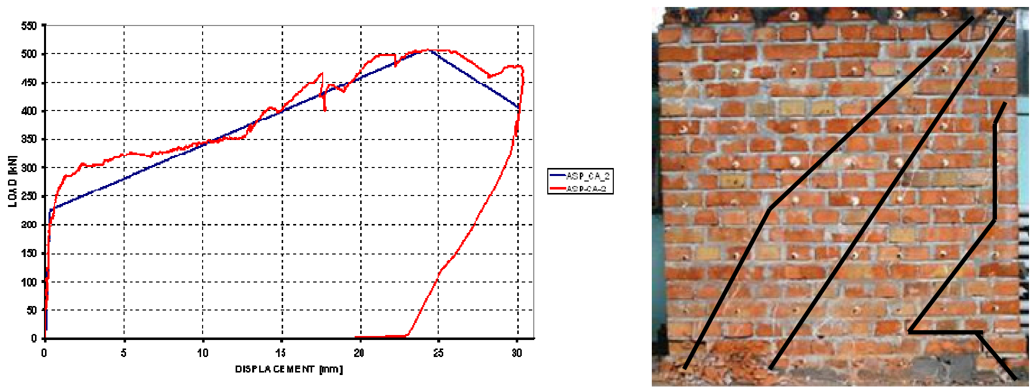
a) Reference specimen



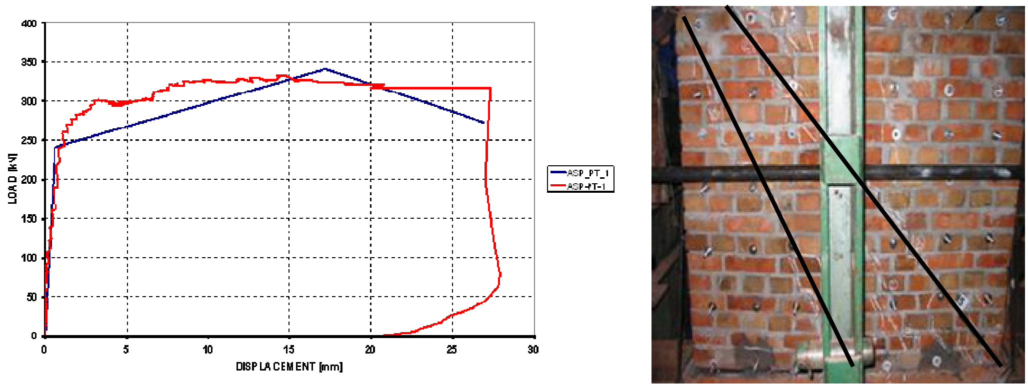
b) ASP – CA - 1m



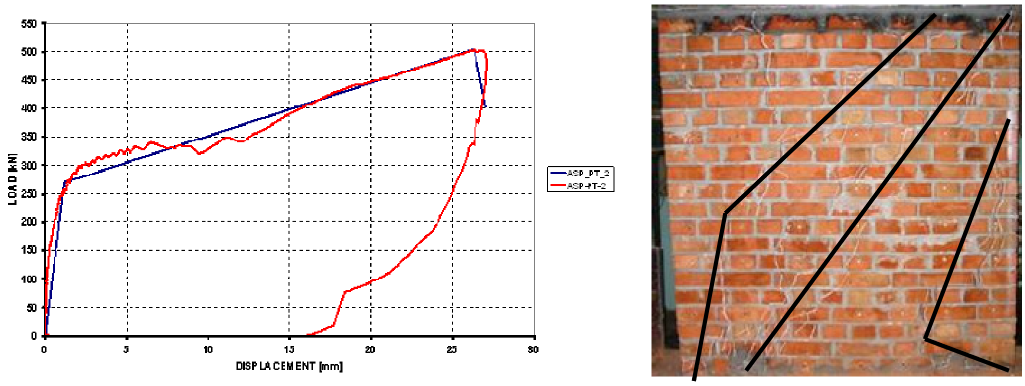
c) ASP – CA - 2m



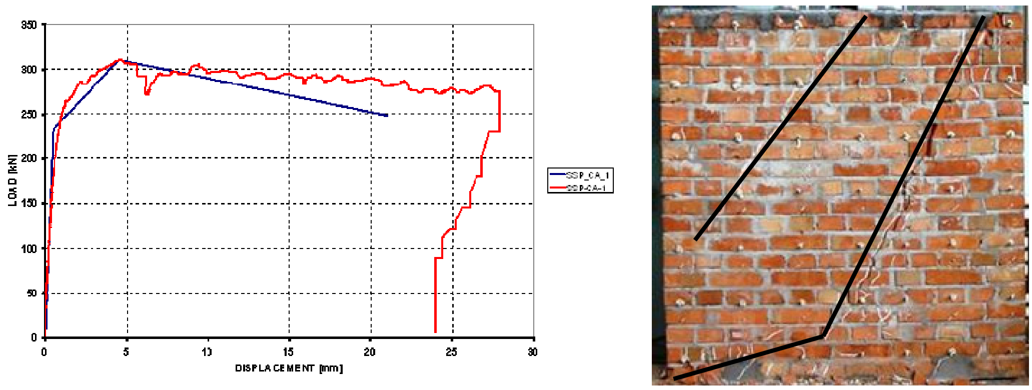
d) ASP – PT – 1m



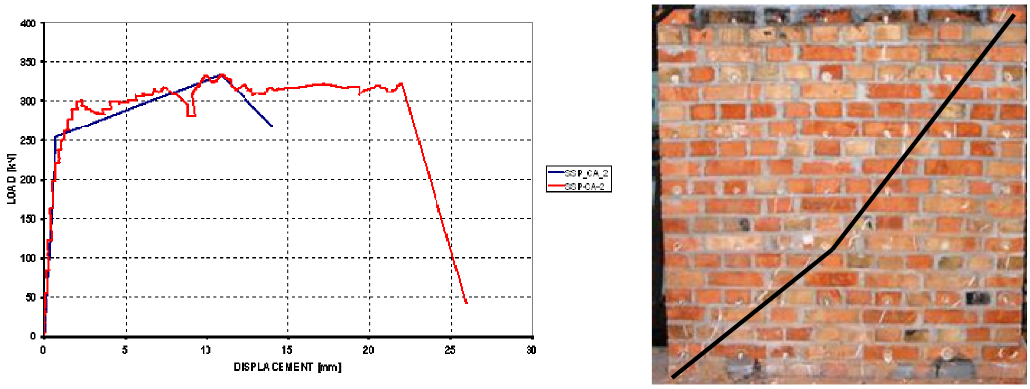
e) ASP – PT – 2m



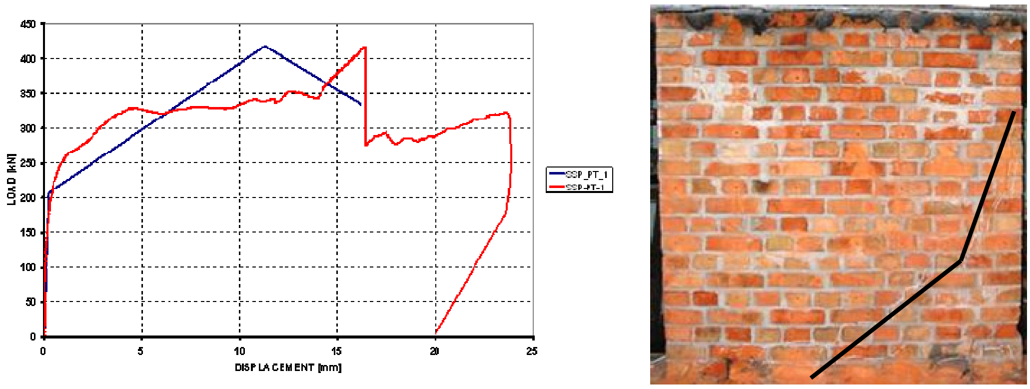
e) SSP – CA – 1M



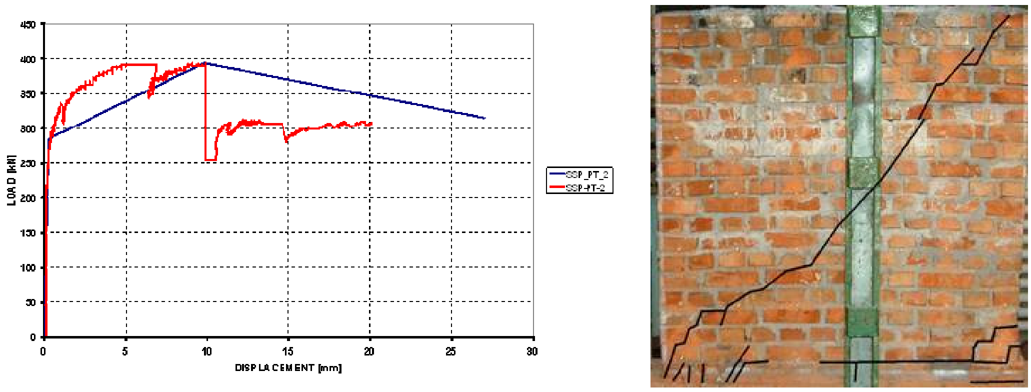
f) SSP – CA – 2M



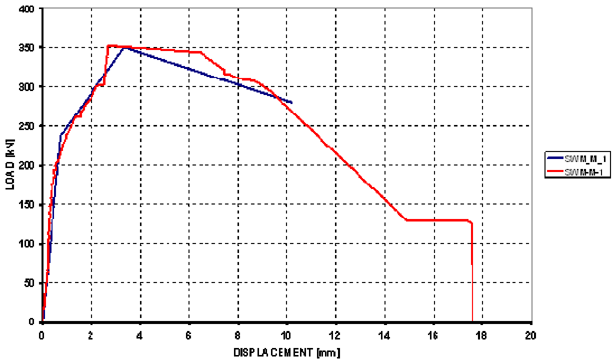
g) SSP – PT – 1M

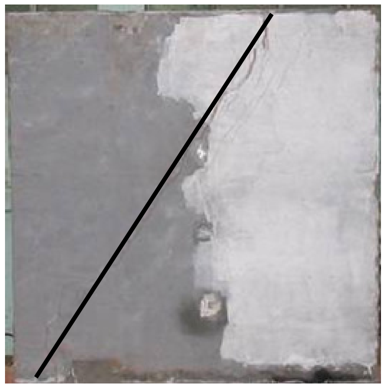


h) SSP – PT – 2M



i) SWM – 1M





j) SWM – 2M

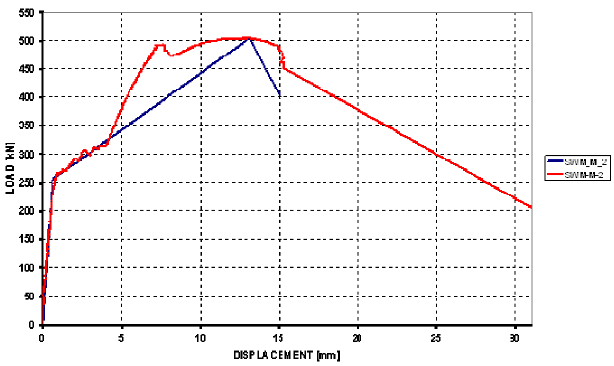
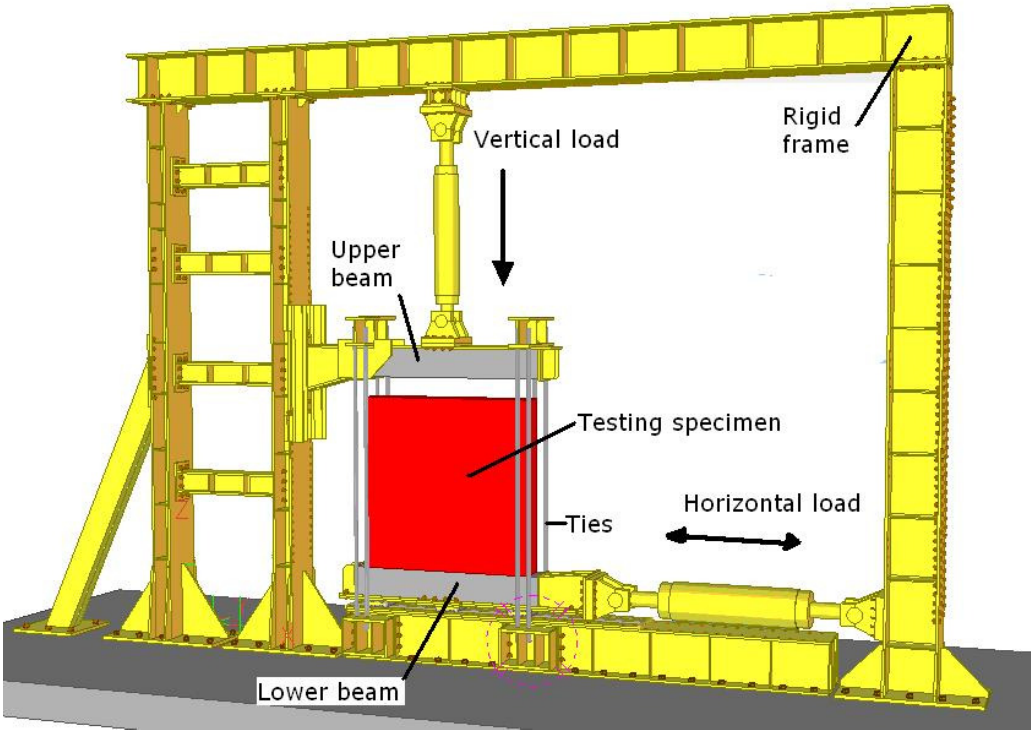


Figure 4-30. Experimental behaviour curves and failure mode for monotonic specimens

4.6.3. Testing of large specimens under cyclic conditions

The test set-up for **cycling** loading consists in the composite beams steel – concrete and was specially designed for this experimental program. The testing specimens is put between the beams and glued with mortar. The upper beam considers restrain as a fixed base and the lower beam is simply supported having only the possibility to slide horizontally without rotations possibilities. The set-up and displacement of transducers is presented in Figure 4-31.

The plotted results are shown as the pure shear deformation vs. force and the failure mode are also presented. Because of the large stiffness of the testing set-up which gives significant rotation of the upper beam was chosen to plot only the pure shear deformation.



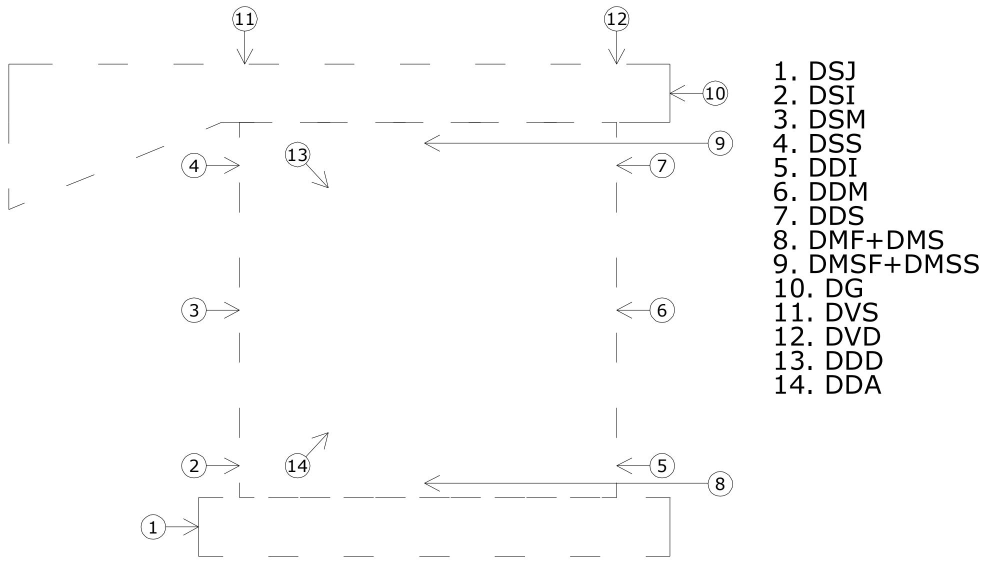


Figure 4-31. Testing frame for cyclic loading

The tests were carried-out in two steps (see Figure 4-32). First there has been applied a vertical load of 30t, after the upper reinforced concrete - steel beam was restrained and the lateral steel bars were tight.

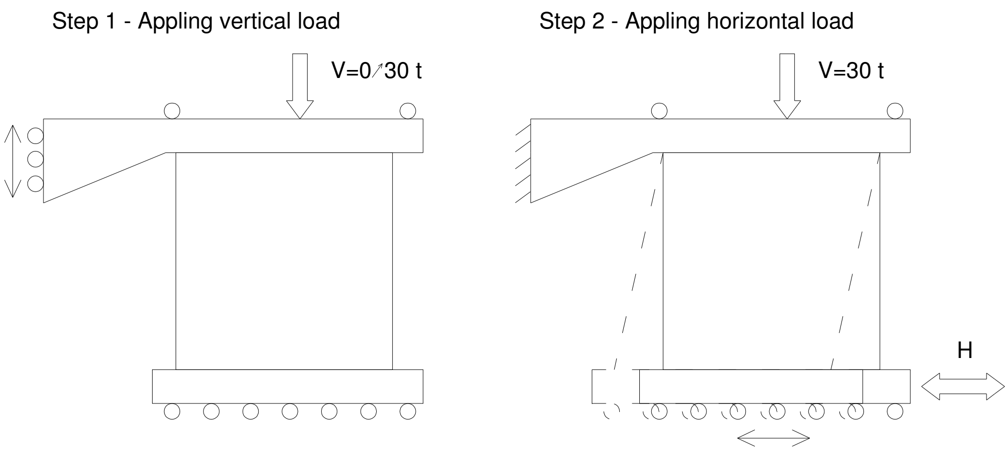


Figure 4-32. Steps for cyclic loading

Loading was applied using displacement control, with the lateral drift of the panel being used as control parameter. In the case of cyclic loading the following loading protocol was used: one cycle at ±0.5 mm, ±1.0 mm, ±1.5 mm, ±2.0 mm, ±3.0 mm, ±5.0 mm, ±7.0 mm, ±9.00 mm, ±11.00 mm, etc. The "yield" displacement, ey, was considered when significant stiffness degradation was observed. After "yielding", three cycles at ey, 1.5 ey, 2 ey, etc. were applied, until the failure of the specimen occurred. For SSP specimens was applied the displacement related to (DDI-DDS) displacement recording and for ASP,SWM and REF related to (DMF+DMS)/2-(DMSF-DMSS)/2 displacement formula.

The plotted results are shown the shear deformation vs. force. The obtained pure shear deformation was obtained from the diagonal displacement transducers, DDA, respectively DDD, as follow:

$$\gamma = \frac{\sqrt{a^2+b^2}\left(DDA-DDD\right)}{2ab} \Rightarrow D_{shear} = \gamma \cdot h$$

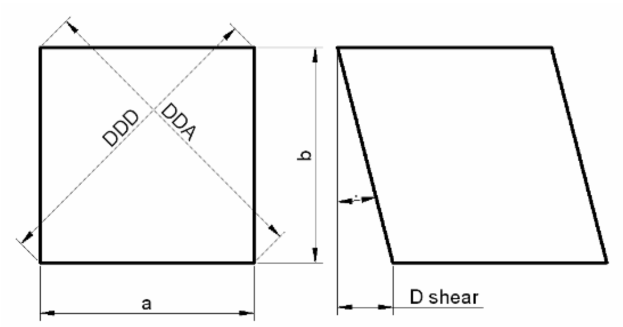
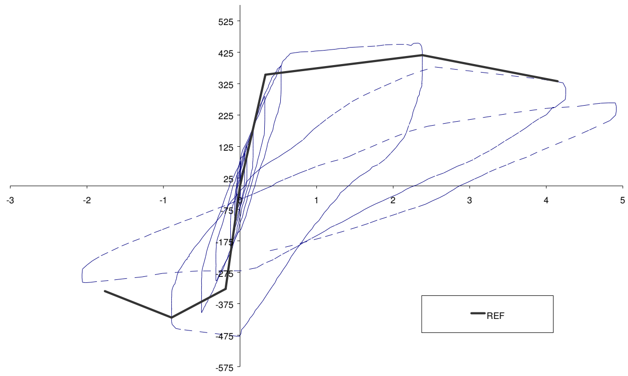
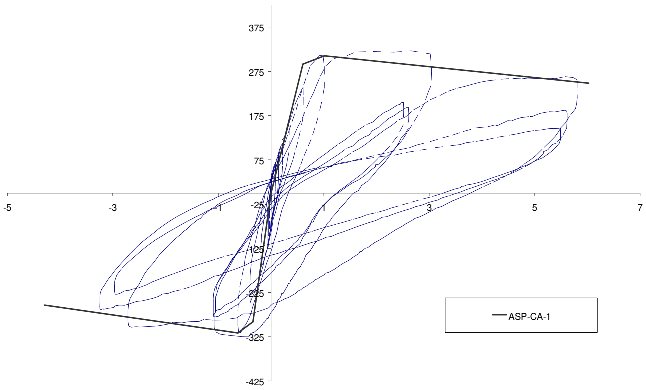


Figure 4-33. Diagonal displacement transducers and shear deformation

Reference



ASP – CA – 1C



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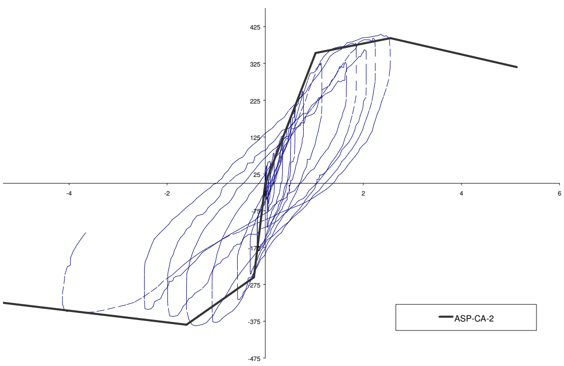
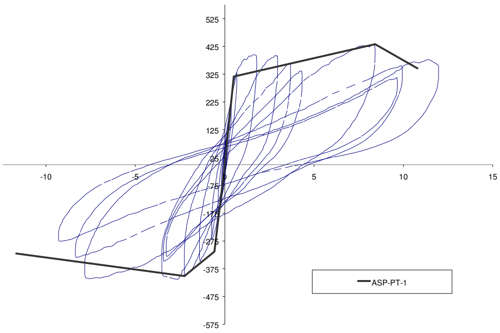
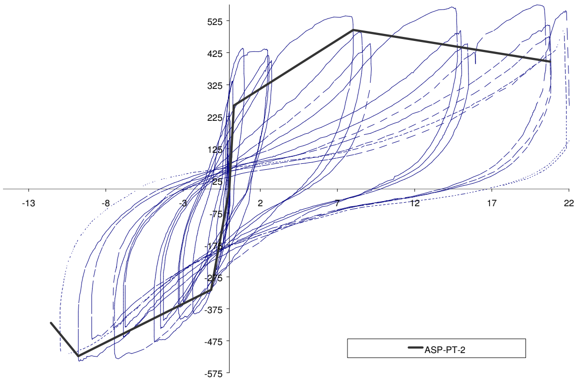


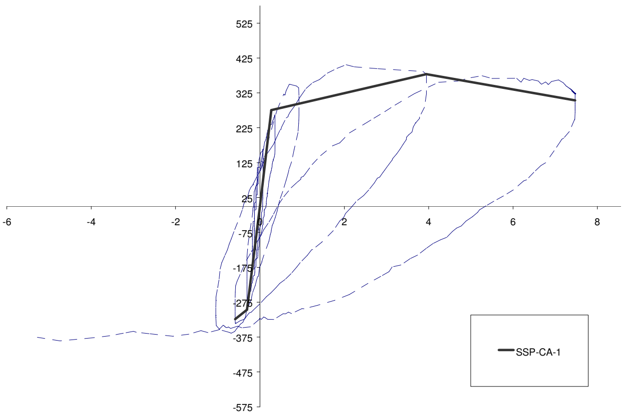
ASP – PT – 1C



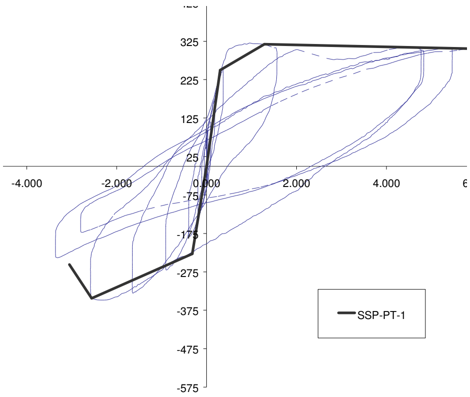
ASP – PT – 2C



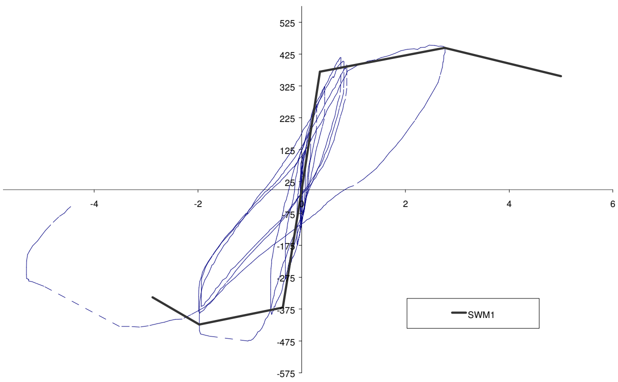
SSP – CA – 1C



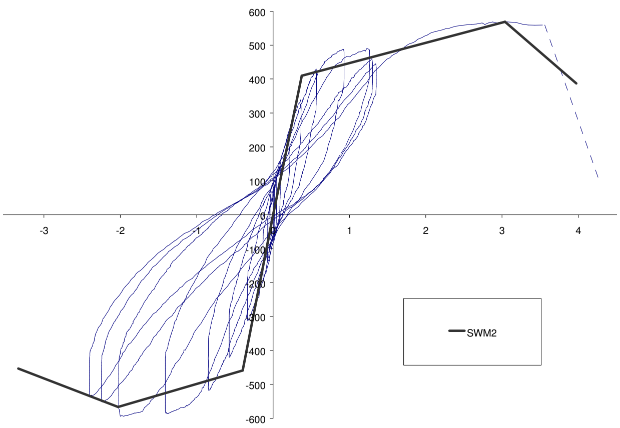
SSP – PT – 1C



SWM – 1C



SWM – 2C



BUPT

SSP – PT – 2C
N.A.*

SSP – CA – 2C
N.A.*

* In case of SSP-CA/PT applied on both sides the displacement transducers DDD and DDA were attach directly to the sheating recording the shear deformation of the steel plates and not of the masonry walls.

Figure 4-34. Experimental behaviour curves and failure mode for cyclic specimens

4.6.4. Conclusions

The diagonal failure mode was observed for all specimens, both under monotonic and cyclic loading. There were also observed some horizontal hairline cracks in the bed joints at the heel of the wall, together with the crushing of wall corners. At some specimens, mainly at the ones retrofitted only on one side, some out-of-plane movement at the top and bottom parts have been recorded. All these failure mechanisms presented above prove that the retrofitting systems have forced the masonry wall to activate ost of its strength and deformability capacity.

Due to the flexibility of the testing frame used for cyclic loading, a more substantial damage at the corners of the panel was observed in comparison to monotonic tests. However, for cyclic loaded specimens the characteristic failure was also by diagonal shear with an influence of eccentric compression. A significant improvement in terms of ultimate displacement (that shows significant improvement in ductility), and also the increase in strength, with a slight increase in stiffness were recorded for all specimens. An overview of performance in terms of strength and ductility of tested specimens, related to unreinforced masonry, is presented in Table 4-12 and Table 4-11. In the following figures Figure 4-35 and Figure 4-36, only the pure shear behaviour of the masonry walls is presented.

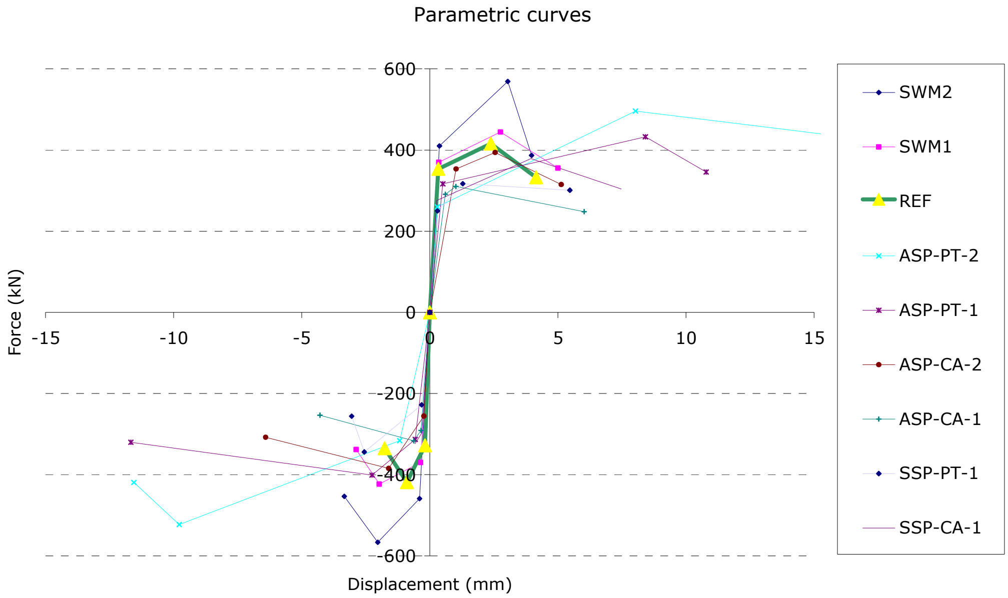


Figure 4-35. Shear deformation - parametric curves for cyclic specimens

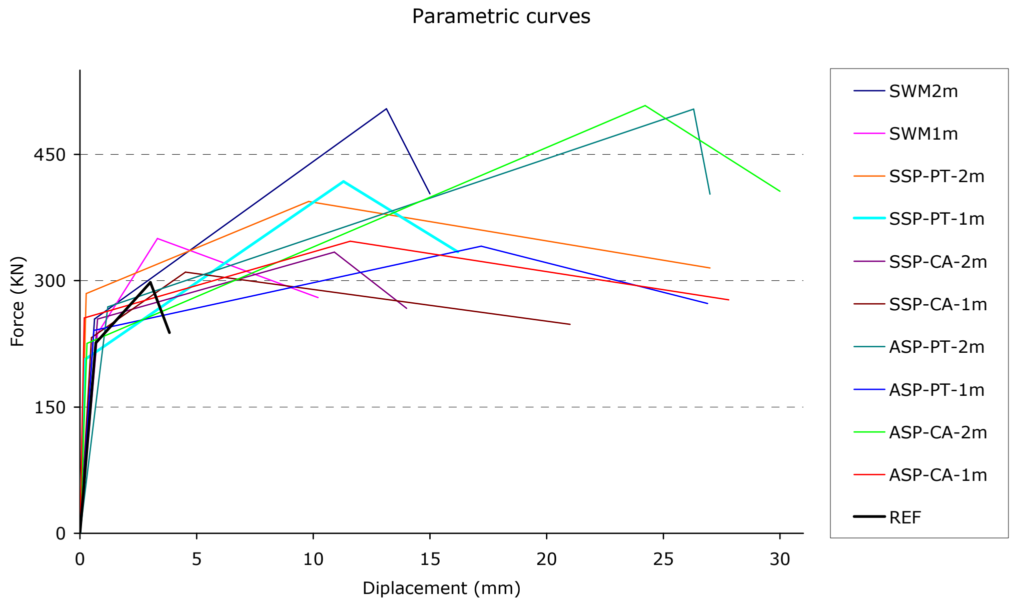


Figure 4-36. Parametric curves for monotonic specimens

For the one side sheeting under cyclic loading a significant out of plane deformation was observed.

Also for cyclic loading this system has proved its validity by increasing the resistance and obtaining a good hysteretic behaviour despite significant pinching.

The proposed strengthening solutions are an alternative to FRP technology enabling to obtain a ductile increase of strength, but without increasing the stiffness of the wall. It can be concluded that SP increases mainly the ductility, while WM increases the resistance. Both techniques are more efficient when applied on both sides. The prestressed tie connections seem to be more appropriate and the specimens sheeted with aluminium plates have shown a better behaviour than ones sheeted with steel. The proposed strengthening systems, which are all innovative, were confirmed.

Table 4-10. Large specimens' qualitative results

	Monotonic		Cyclic	
	Resistance	Ductility	Resistance	Ductility
ASP–CA-1	→	↗	↗	↗
ASP–CA-2	↑	↑	↗	↗
ASP-PT-1	↗	↗	↗	↗
ASP-PT-2	↑	↑	↑	↑
SSP–CA-1	→	↗	→	↗
SSP–CA-2	→	→	→	↗
SSP-PT-1	↗	↗	→	↗
SSP-PT-2	↗	↑	↗	↗
SWM-1	→	→	↗	→
SWM-2	↑	↗	↑	↗

Legend → slightly ↗ moderate ↑ high increase

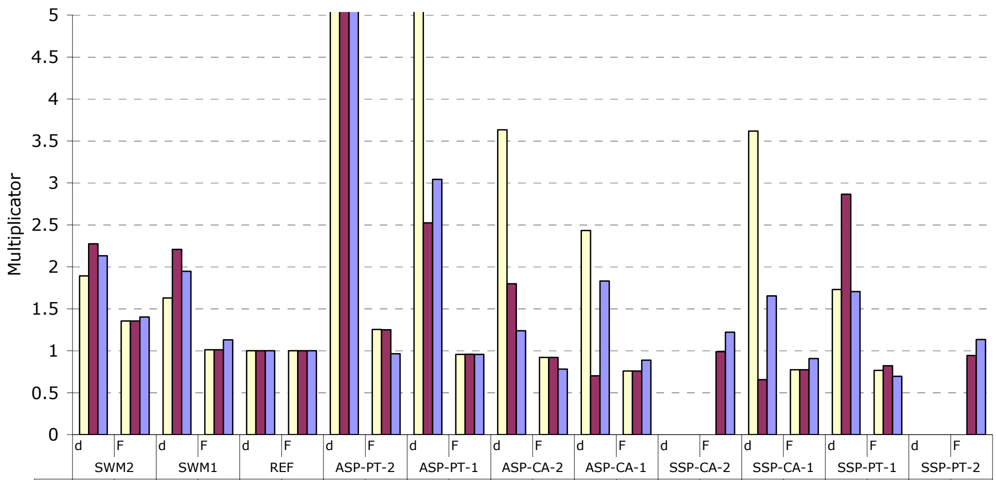
Due to the high scattering of the masonry mechanical characteristics, in order to perform a statistical interpretation of the experimental results are needed many experimental specimens on the same techniques. In our case were experimentally study 10 different techniques for masonry walls retrofitting and only one specimen for each of these. On this observation, the experimental results have more an qualitative values offering an orientative results values for the ultimate strength and the ultimate displacement of the retrofitted walls (see Table 4-10).

It is expected that these strengthening solutions should also be applied for weak reinforced concrete diaphragms.

Figure 4-37 shows the parametric curves for cyclic loading specimens. As displacement, is plotted the lateral absolute displacement of the panel included the pure shear deformation and also the rocking of the wall. Because in the upper beam there was recorded significant rotation (in the extremities there have been recorded vertical displacements comparable with the horizontal displacements) these parametric curves are not very reliable, but they show the displacement increase in the case of the combined failure mechanism.

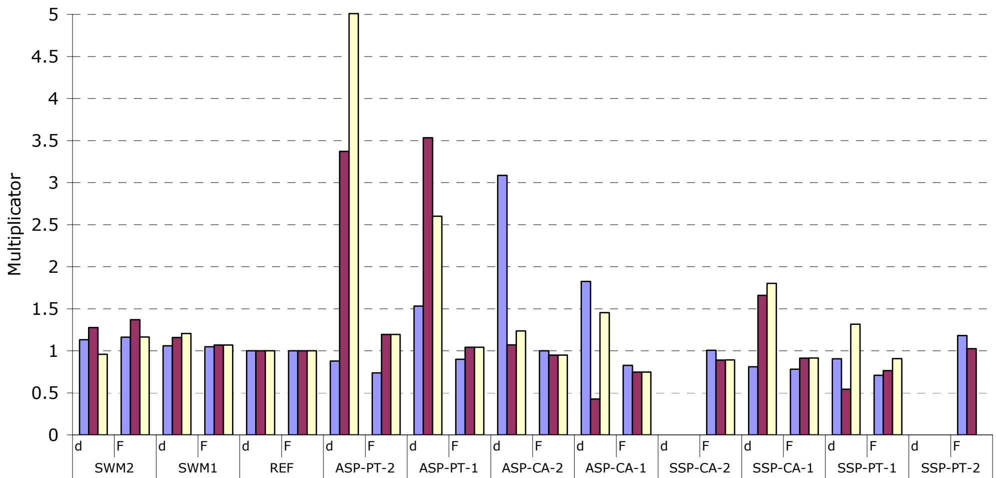
Table 4-11. Comparative results on large specimens tested in cyclic loading refer to reference

Comparative negative cycles results / reference specimen



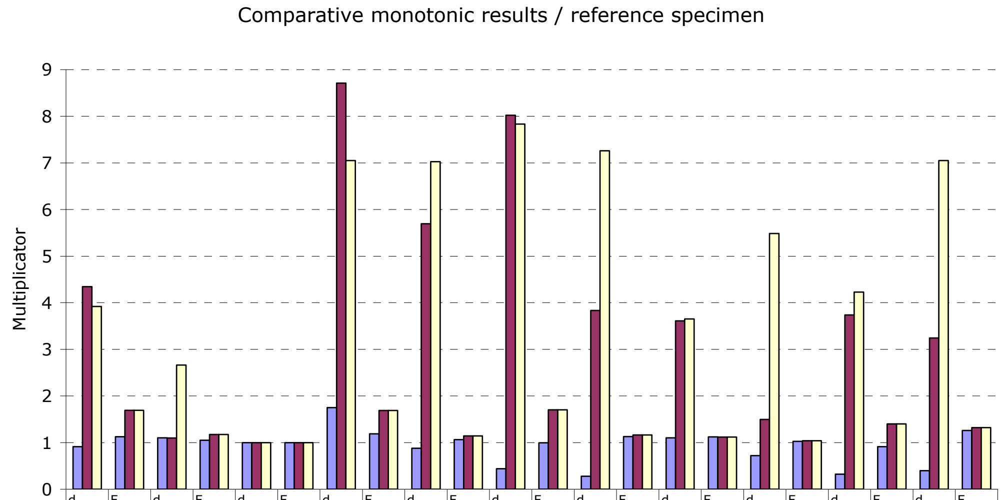
	SWM2 d	SWM2 F	SWM1 d	SWM1 F	REF d	REF F	ASP-PT-2 d	ASP-PT-2 F	ASP-PT-1 d	ASP-PT-1 F	ASP-CA-2 d	ASP-CA-2 F	ASP-CA-1 d	ASP-CA-1 F	SSP-CA-2 d	SSP-CA-2 F	SSP-CA-1 d	SSP-CA-1 F	SSP-PT-1 d	SSP-PT-1 F	SSP-PT-2 d	SSP-PT-2 F
U-	1.89	1.353	1.629	1.009	1	1	6.545	1.251	6.609	0.955	3.631	0.919	2.431	0.756	0	0	3.617	0.774	1.728	0.764	0	0
M-	2.274	1.353	2.208	1.011	1	1	10.95	1.249	2.522	0.958	1.797	0.919	0.701	0.756	0	0.988	0.654	0.774	2.863	0.822	0	0.941
E-	2.13	1.4	1.946	1.129	1	1	6.283	0.964	3.04	0.956	1.237	0.78	1.83	0.888	0	1.22	1.651	0.905	1.705	0.695	0	1.132

Comparative pozitive cycles results / reference specimen



	SWM2 d	SWM2 F	SWM1 d	SWM1 F	REF d	REF F	ASP-PT-2 d	ASP-PT-2 F	ASP-PT-1 d	ASP-PT-1 F	ASP-CA-2 d	ASP-CA-2 F	ASP-CA-1 d	ASP-CA-1 F	SSP-CA-2 d	SSP-CA-2 F	SSP-CA-1 d	SSP-CA-1 F	SSP-PT-1 d	SSP-PT-1 F	SSP-PT-2 d	SSP-PT-2 F
E+	1.131	1.16	1.059	1.047	1	1	0.878	0.736	1.532	0.896	3.085	1	1.823	0.824	0	1.003	0.808	0.78	0.904	0.708	0	1.179
M+	1.276	1.369	1.157	1.069	1	1	3.37	1.193	3.534	1.041	1.071	0.948	0.426	0.746	0	0.89	1.658	0.912	0.541	0.763	0	1.024
U+	0.957	1.164	1.205	1.069	1	1	5.008	1.193	2.6	1.04	1.236	0.948	1.452	0.746	0	0.892	1.8	0.913	1.317	0.905	0	0

Table 4-12. Comparative results on large specimens tested in monotonic loading refer to reference



	SWM2m d	SWM2m F	SWM1m d	SWM1m F	REF d	REF F	ASP-PT-2m d	ASP-PT-2m F	ASP-PT-1m d	ASP-PT-1m F	ASP-CA-2m d	ASP-CA-2m F	ASP-CA-1m d	ASP-CA-1m F	SSP-CA-2m d	SSP-CA-2m F	SSP-CA-1m d	SSP-CA-1m F	SSP-PT-1m d	SSP-PT-1m F	SSP-PT-2m d	SSP-PT-2m F
E	0.912	1.126	1.103	1.05	1	1	1.75	1.189	0.882	1.065	0.441	0.997	0.279	1.131	1.103	1.125	0.721	1.027	0.324	0.914	0.397	1.26
M	4.348	1.693	1.099	1.175	1	1	8.709	1.691	5.695	1.145	8.02	1.705	3.834	1.164	3.609	1.121	1.497	1.041	3.738	1.402	3.245	1.322
U	3.916	1.693	2.663	1.175	1	1	7.05	1.691	7.023	1.145	7.833	1.705	7.258	1.164	3.655	1.121	5.483	1.041	4.23	1.402	7.05	1.322

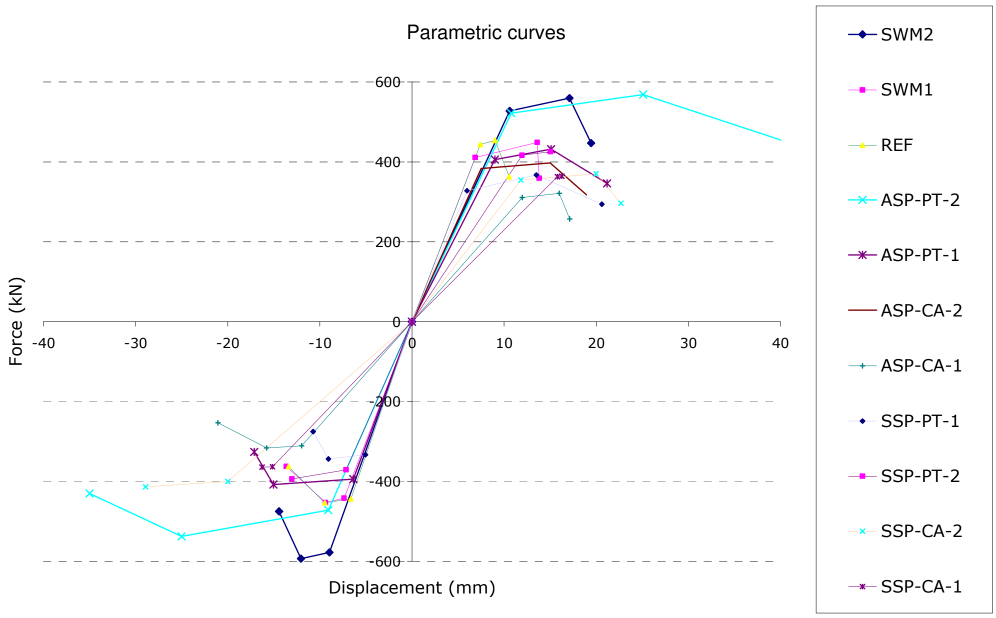


Figure 4-37. Lateral displacement - parametric curves for cyclic specimens

A relevant comparison between monotonic and cyclic loading condition specimens is hard to make because by changing the testing set-up important changes occurs and it is difficult to quantify the particular effects on the wall behaviour, but the same benefit effects of the retrofitting techniques are still shown in essence.

No particular difficulties for practical application in comparison with other used techniques have been observed.

Due to large in-plane stiffness of masonry walls, as was expected the strengthening solution does not completely avoid damage to masonry. A limited amount of damage to masonry has to be allowed in order to take benefit from the ductility of the metal used for sheathing. It can be observed that, despite strengthening, the masonry panel cracks at almost the same force and displacement values as pure masonry reference panel. The mixed masonry-metallic plate work is activated only after masonry cracking. This can be observed also by the fact that the initial stiffness of both strengthened and reference panels don’t change too much. This is an advantage for the global behaviour of retrofitted building. These observations emphasize that the system doesn’t modify the life safety performance level. The major advantage of these techniques, experimentally confirmed, is the important increase, of 3-5 times, of the ultimate displacement accompanied by increase of load bearing capacity of the retrofitting masonry panels in comparison to the reference unreinforced panel. This fact ensures a stable post-crack behaviour allowing to establish a new performance point in the case of CP performance level of the element (see Figure 6-5).

Based on this observation there can be established the beneficial effect of the reinforcing techniques that enhance ductility and ensure strength demand in the range LS - CP (see Figure 6-5), and, very important, allow a better prediction of structure behaviour during earthquake.

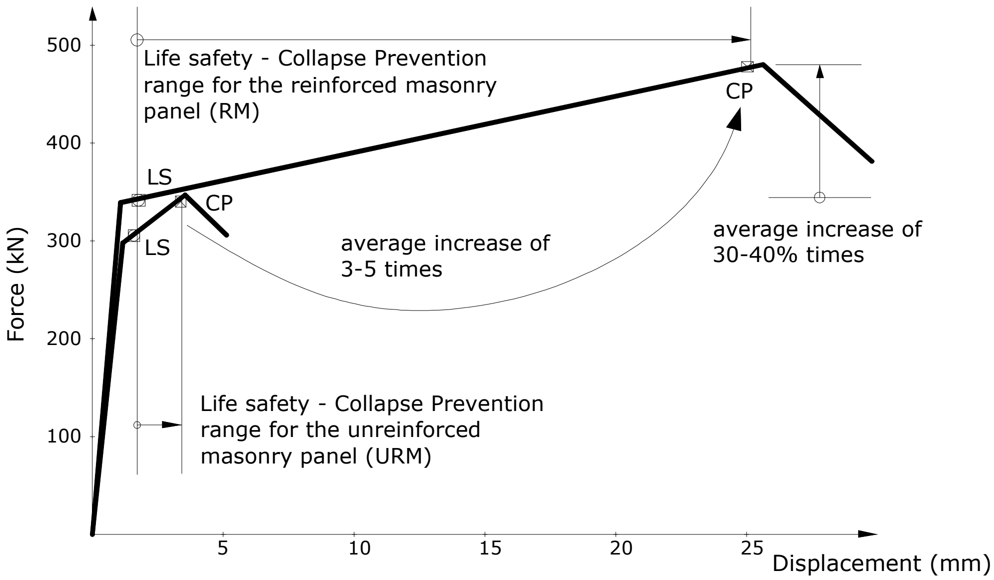


Figure 4-38. Benefit of the reinforcing from the performance levels point of view

Once again it is important to underline that the presented results mainly focus on shear behaviour (in the case of cyclic loading specimens) and, due to the combined failure mechanism, the displacement of the walls is much more significant. Also having in mind the rocking of the walls and the failure of the cyclic tested specimens due the crushing of the wall toe, by supplementary measurements to reinforce this area or by placing rubber elements, much more important improvements can be obtained. Also by a proper connection of the metallic sheathing with adjacent elements, it is expected that the positive effect should increase a lot.

5. CHARACTERIZATION AND EVALUATION OF STRUCTURAL PERFORMANCE OF VIRGIN AND STRENGTHENED MASONRY

5.1. INTRODUCTION

This chapter will especially treat the behaviour of the masonry structures made by clay bricks and the cement mortar.

Masonry is one of the most ancient ways to build. The blocks, the mortar, the construction techniques were always adapted by local masons, depending on the historical age and place. Locally available materials and local experience, had a major influence on the local masonry structures. Although at the end of the last century some standardization could be recorded, most of the older masonry buildings are very characteristic to their location. In the Mediterranean aria there is a wide range of masonry elements in terms of pattern and also in terms of composing materials, like adobe, stone with lime – mortar etc. The most common masonry wall typologies are: Single leaf walls; Cavity walls with rubble filled core; Bonded brickwork walls; Stone masonry walls; Walls made of lightweight Concrete Masonry Units (CMU); Concrete block walls; More special masonry wall configurations: Pombalino walls - Timber and masonry combination developed after the 1755 Lisbon earthquake; Versions of Roman masonry walls clad in the most varied configurations (e.g. opus reticulatum, opus testaceum, opus mixtum, opus craticium, opus quadratum, etc).Tuff masonry constructions – Typical to earthquake locations in Southern Italy, Turkey, Armenia etc. [129].

The blocks commonly used in masonry walls can be: (a) adobe, (b) solid clay brick, (c) cellular clay brick, (d) hollow clay brick, (e) perforated clay bricks, (f) hollow concrete blocks, (g) cellular concrete blocks, (h) autoclaved cellular concrete blocks, (j) stone blocks (with blocks in different processing state), etc.

The mortar used in masonry construction is as varied as the blocks themselves. In new constructions the most common mortars are: Portland Cement (PC) mortar, calcium mortar or mixed calcium and cement mortars. But most of the walls can be constructed without the use of mortar altogether.

All this typologies of masonry respect more or less the same principles in terms of behaviour.

In this chapter, are overviews the main approaches for numerical investigations of virgin masonry and the proposed innovative retrofitting system for masonry shear walls are presented. In order to set up an appropriate and reliable design tool, a numerical model has been developed using the non-linear software Abaqus.

The possibilities to calibrate and design such technique, like the ones proposed in this thesis, are limited from the analytical or calculation methods points of view because until now there are no specific provisions for this type of intervention or for similar ones. Therefore the design of the consolidation system can be based either on experimental tests or on the available finite element models able to simulate the real behaviour of the masonry – steel composite system. The latter desiderate is not easy to be acvieved, since also for a very simple masonry panel, a good accordance between the real behaviour of the panel and the numerical results is difficult to obtain.

A complex problem like the evaluation of the bearing capacity of a structure and the decision regarding the necessity of a structural intervention must start from the study and thorough understanding of the real nature and stress-strain behaviour of materials and elements behaviour.

Masonry is the oldest building material (Figure 5-1) that still finds wide use in today's building industries and remains one of the most used construction materials. The most important advantage of masonry construction is its simplicity, but in the same time a lot of difficulties arise when it comes to evaluating its behaviour. When a masonry building is the subject of strengthening, the strength technique cannot avoid considering the base material behaviour, and modelling. These paragraphs make a review of the analytical formula of masonry behaviour, as a composite material, in different loading condition through advanced numerical possibilities. The focus is on the shear behaviour of masonry panels only, without treating the out-of-plain behaviour, and elements like arches and vaults.

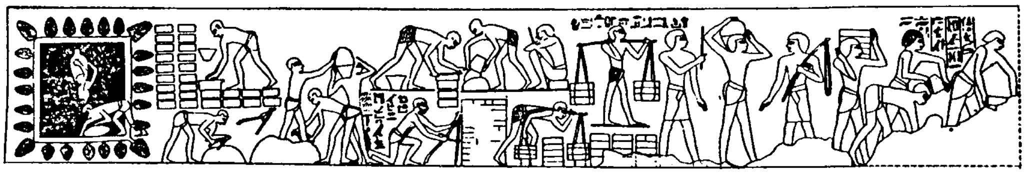


Figure 5-1 Brick making in Egypt (wall painting in the tomb of Rekhmara at Thebes 1500 BC).

5.1.1. Models for masonry component materials

In the case of a composite material like masonry the first step for calculation and analysis requires an exact study of the component material – material model. This chapter will describe the behaviour of clay brick and cement mortar. Material tests on single brick blocks and mortar specimens (Figure 5-2) are needed in order to establish the behaviour of each component.

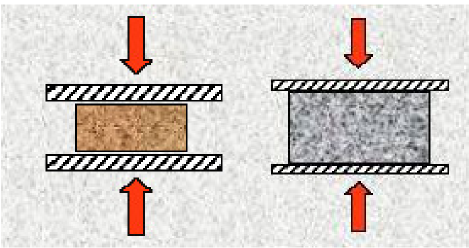


Figure 5-2 Compression tests for brick/mortar units

Usually, this test offers a behaviour curve from which we can extract the load bearing capacity and deformation characteristics. These tests give us the physical models for each material. In Figure 5-3 shows the physical models for brick and mortar in case of uniaxial load.

We can draw some conclusions about material behaviour. The component materials have much higher resistance in compression than in tension (6-10 times more); in generally brick has higher elastic modulus and resistance than mortar; in comparison with brick, which shows have a brittle failure, the mortar has a higher ductility and admits higher deformations. By combining these two materials with very different mechanical characteristics a composite and inhomogeneous material is obtained.

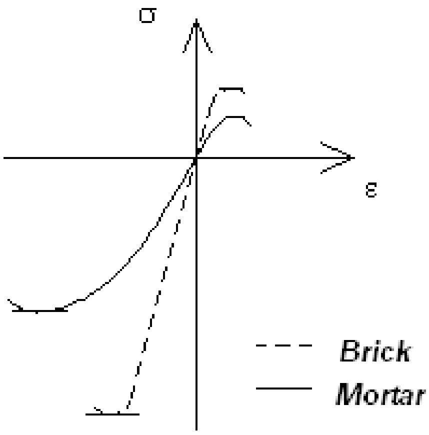


Figure 5-3 Behaviour curves for brick and mortar

Don’t to neglect as a distinctive component of masonry is the bond between units and mortar layer. This interface control the nonlinear response of the joint which is one of the most relevant feauture of masonry behaviour [119].

5.1.2. Models for masonry as a composite material

In this paragraph, we will summarizarize the possibilities to take into account a masonry wall subjected to in-plane loading. The ways to design and analyze an element are presented in a formally ranked order from simple formulas to complex numerical approaches.

5.1.2.1. Design code relationship (Analytical Models)

From the practical point of view of design, the national standards offer different formulas for each failure mode. For example, the old Romanian Code for Masonry Building P2/85 and EC6 [64] suggest simple mathematical relations (Table 5-1) for establishing the resistance of an element.

Table 5-1 Design formulas

Sliding failure	
Small eccentricity $T_{CF} = \frac{A_i}{\mu_i}(R_f + 0.7 f \sigma_0)$ High eccentricity $T_{CF} = \frac{0.7 \cdot f \cdot A_i \cdot \sigma_0}{\mu_i}$	$F_{Rd1}(zu) = \frac{1}{\cos\theta} f_{vd0} l_p t_p (1+\alpha)$ $\alpha = 0.07\left(4\frac{h_p}{l_p} - 1\right)$
Flexural failure	
$T_{CM} = \frac{M_c}{Z} = \frac{Ne_0}{Z} = \frac{1.25RS_c}{Z}$ $S_c = A_c e_0$ $A_c = \frac{N}{1.25R}$	$F_{Rd2}(zu) = 0.8 f_d \cos^2\theta \sqrt[4]{\frac{E_b}{E_z} I_{st} h_p t_p^3}$
Shear failure	
$T_{CP} = \frac{R_p A_i}{\mu_i}\sqrt{1 + 0.8\Phi\frac{\sigma_0}{R_p}}$	$F_{Rd3}(zu) = \frac{f_{vd0} l_p t_p}{0.6\cos\theta}$
P2/85	EC6 [64]

The minimum value from associated resistances gives the most possible failure mode and the correspondent capacity.

Similar relations are given in most standards for masonry buildings available in the world.

This kind of approach is the easiest one, but it only gives information about the resistance of the element, resistance that usually is highly under-evaluated.

Some other codes offer more detailed information about masonry behaviour, like constitutive laws, rigidity characteristic, deformation capacity, performance criteria etc.

The following figure (Figure 5-4) presents, from a qualitative point of view, the uni-axial behaviour of the masonry compared to the component materials.

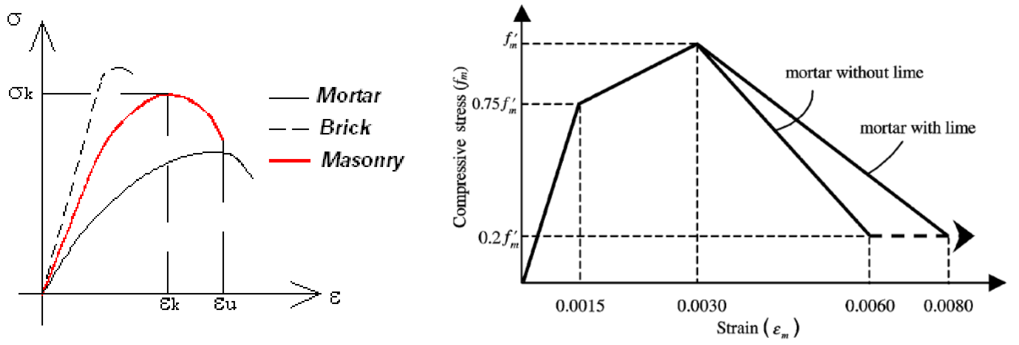


Figure 5-4 Behaviour curves for masonry and its components (uni-axial loading) and simplified tri-linear stress–strain model for masonry [109]

Based on component mechanical properties, Hilsdorf [94] has proposed a relation in order to determine the masonry properties:

$$f_w = 0.9\frac{f_b(f_{bt}+\alpha f_m)}{U(f_{bt}+\alpha f_b)}$$

$$\alpha = \frac{h_m}{4.1h_b}$$

$$U = 2 - \frac{f_m}{34.5}$$

f_m - mortar compression strength

f_b - brick compression strength

f_{bt} - brick tensile strength

h_m - mortar joint depth

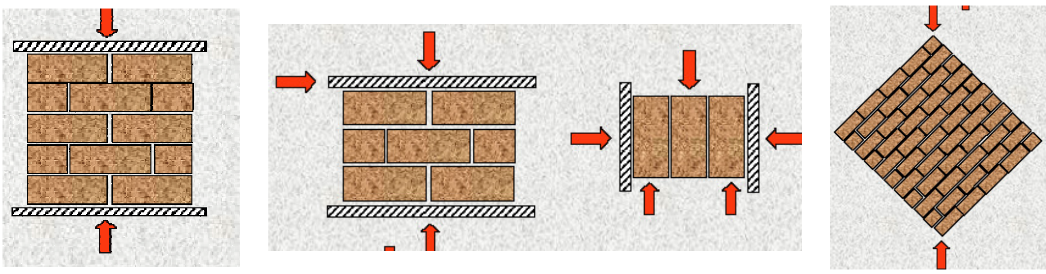
h_b - brick depth

U – non-uniform stress distributing factor

Many national standards give the modulus of elasticity of masonry depend on masonry compression strength. In case of elastic modulus is still a lack of consensus as appropriate relationships. (Prisley and Pauly)

5.1.2.2. Simplified relations for numerical analysis – considering masonry as an homogenous and isotropic material (Analytical Models)

The formulas available are determined from experimental tests on sub-assemblages (Figure 5-5):



Compression test Shear test

Figure 5-5 Usual tests on sub-assemblages

i. Uniaxial loading

Different empirical equations are suggested in scientific papers and standards as constitutive laws for masonry elements as a homogenous material, subjected to compressive loading normal to the bed joints (Table 5-2).

Table 5-2 Constitutive laws for masonry

Turnsek-Cacovic (1970) [210]:

$$\frac{\sigma}{\sigma_k} = 6.4\left(\frac{\varepsilon}{\varepsilon_k}\right) - 5.4\left(\frac{\varepsilon}{\varepsilon_k}\right)^{1.17} \tag{13}$$

Sawko (1982) [196] based on Powell-Hodgkinson experimental test [184]:

$$\frac{\sigma}{\sigma_k} = 2\left(\frac{\varepsilon}{\varepsilon_k}\right) - \left(\frac{\varepsilon}{\varepsilon_k}\right)^{2} \tag{14}$$

ANDIL (Italian Association of Clay Brick Producers):

$$\frac{\sigma}{\sigma_k} = 3.4142 \cdot \left[1 - \left(1 + \frac{\varepsilon}{\varepsilon_k}\right)^{0.5}\right] \tag{15}$$

EC 6 propose [64]:

$$\begin{aligned} \frac{\sigma}{\sigma_k} &= 2\left(\frac{\varepsilon}{\varepsilon_k}\right) - \left(\frac{\varepsilon}{\varepsilon_k}\right)^{2} \qquad && for\ 0 \le \varepsilon \le \varepsilon_k \\ \frac{\sigma}{\sigma_k} &= 1 && for\ \varepsilon_k \le \varepsilon \le \varepsilon_u \end{aligned} \tag{16}$$

Legend:

σ_k = maximum allowable compression strength

ε_k = 0.002 (characteristic strain correspondent σ_k)

ε_u = 0.003÷ 0.0045 (ultimate strain)

The most common test for determine the compressive resistance normal to the bed is RILEM test [222]. Mann and Betzler [128] have shown in there papers the influence of the different forms of test samples to the compressive strength of masonry.

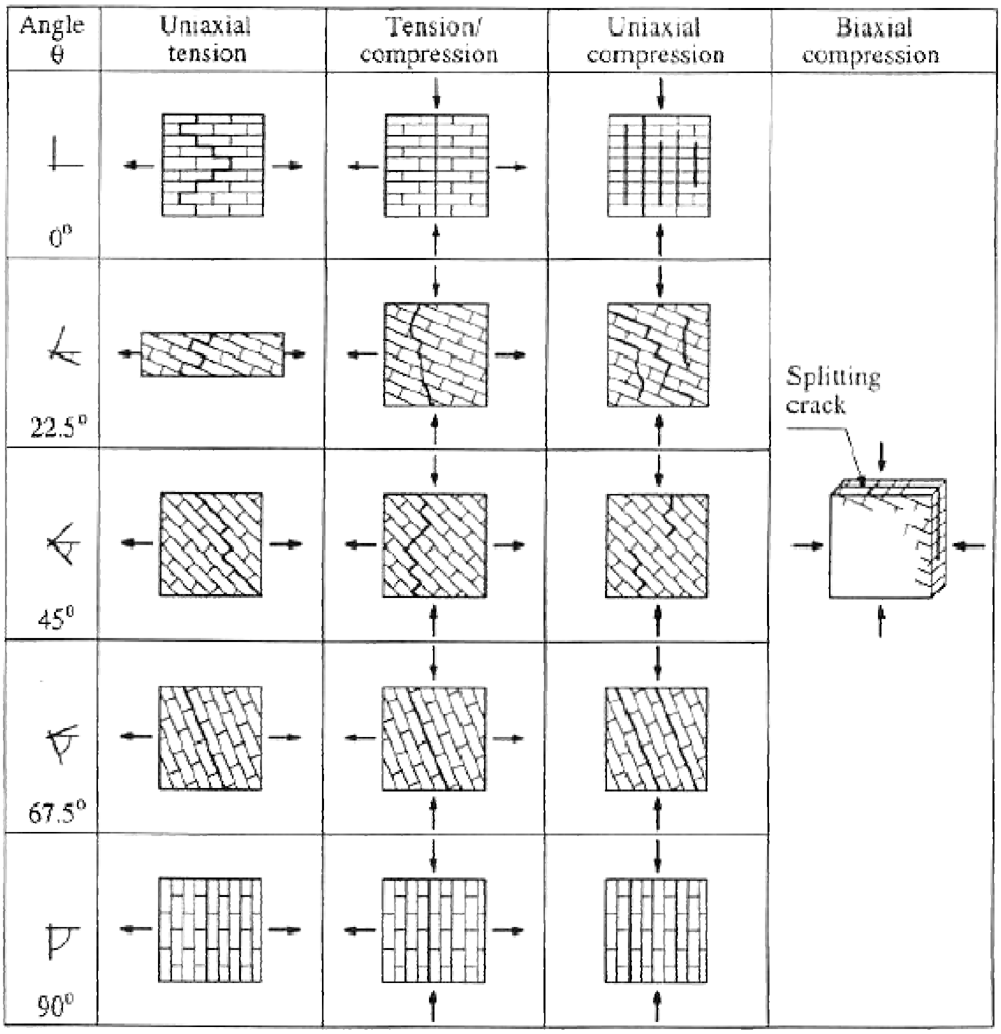
The uniaxial compressive resistence parallel to the bed joints comparative to one normal to the bed joints ranges between 0.2-0.8 [96].

In case of tensile behaviour, masonry exhibits pose a very poor resistance and the behaviour shows an accentuated softening behaviour. Similar to concrete, for masonry elements can be used a parabolic softening law [97]. Usualy the tensile stress is expressed depeding on compressive strength. This fact is not entirely true due to dependency on shape, materials, manufacture process etc [119].

ii. Biaxial loading

Being a inhomogeneous material with different component materials, masonry shows a very different behaviour in relation to the direction of the applied load. Consequently the resistance of the element is highly dependent of the biaxial state of stress in the element, the uni-axial behaviour being insufficient for describing the material behaviour. Failure modes for different biaxial tests specimens are presented the following table (Table 5-3):

Table 5-3 Different biaxial tests [50]



Many experimental works [173] have been carried out and they suggest the following biaxial interaction curve between the principal stresses (Figure 5-6).

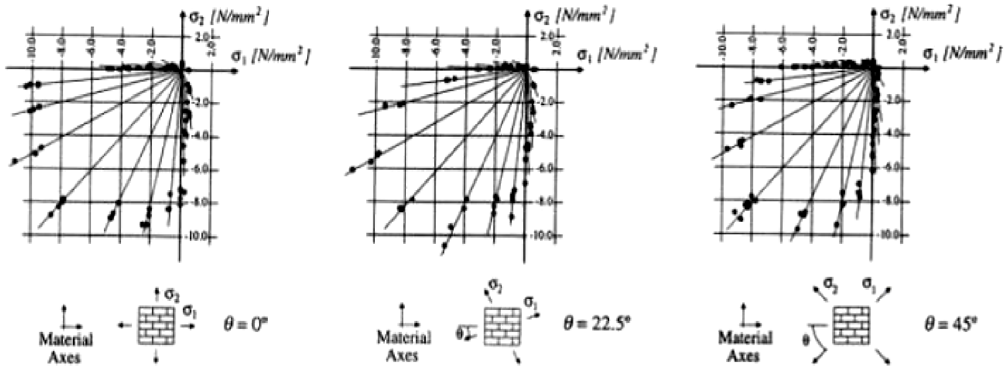


Figure 5-6 Experimental interaction curves [173]

Based on experimental tests, a failure domain (Figure 5-7 – Concrete smeared cracking used by ABAQUS) used for numerical, finite element simulation seems suitable. Also, there are other types like the Hill type [93] and Rankine [121] type composite yield surface [119], Hoffman type single yield surface, yield surface proposed by Dhanasekar [50] or by Ganz [82], or other yield surfaces used mainly in geotechnical numerical simulation.

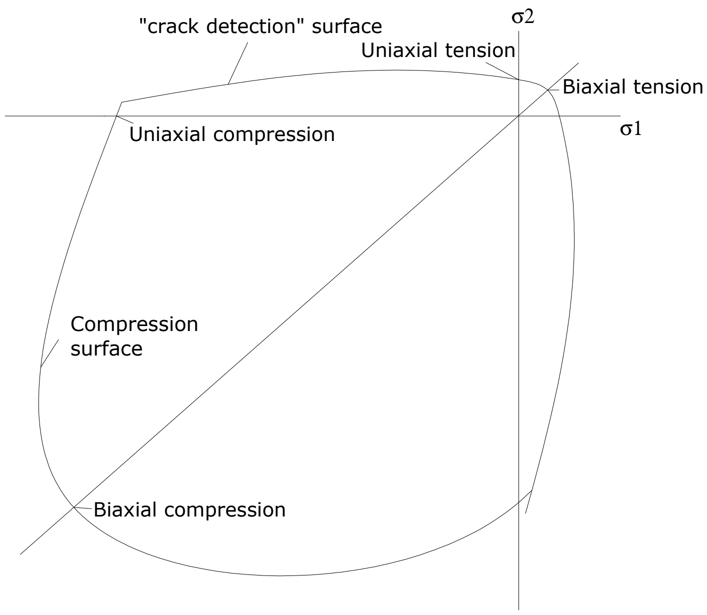
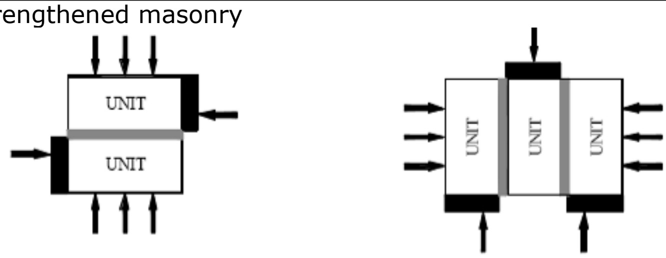


Figure 5-7 Theoretical interaction curves (ABAQUS – Concrete smeared cracking [9])

<u>iii. Shear loading</u>

The most important phenomenon that governs the behaviour of masonry panels is shear behaviour. In order to determine the mechanical characteristics needed for the analysis, some easy tests (Figure 5-8) should be performed on "double" (a) or "triple" (b) samples. The testing set-up is presented bellow:



(a) double sample (b) triple sample

Figure 5-8 Pure shear test

Tests at different levels of normal pre-compression showed that the failure criteria have a Mohr – Coulomb shape. This aspect underlines the importance of the pre-compression level for the behaviour of masonry walls. The tendency of uplifting of masory subjected at shear loading is, also, an important parameter defined by dilatancy angles.

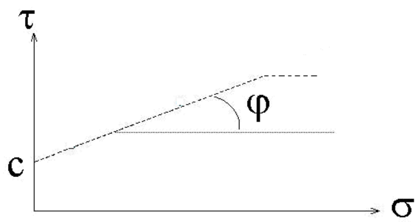


Figure 5-9 Mohr – Coulomb behaviour

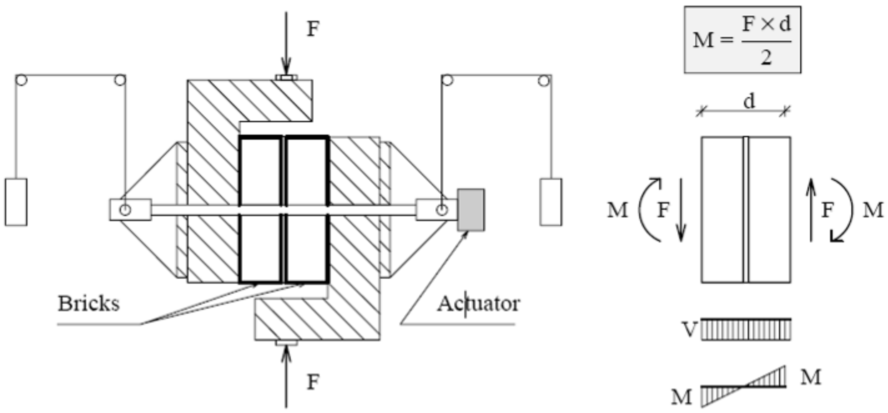
The Mohr – Coulomb relationship is described by the following formula:

$$\tau_{ult} = \tau_0 + \mu\sigma_v$$

τ_0 - ultimate tangential stress at zero level of pre-compression (cohesion)

μ - friction coefficient

σ_v – pre compression stress



a) Test set-up b) Applied forces

Figure 5-10 Apparatus to obtain shear behaviour [183]

In case of masonry behaviour prediction, treated as an homogenous material, other more sophisticated failure criteria like Drucker–Prager–Cap modified [58] or other models that catch the influence of normal state of stress at the ultimate shear capacity, can be used as well.

Knowing the geometry of a panel and the loads applied, the principal stress can be determined with the following relationships:

$$\sigma_c = \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + \left(\frac{b}{\tau}\right)^2} + \frac{\sigma_0}{2} \tag{17}$$

$$\sigma_t = \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + \left(\frac{b}{\tau}\right)^2} - \frac{\sigma_0}{2} \tag{18}$$

Where $\sigma_0 = \frac{V}{A_w}$ the average compressive stress due to vertical load; $\tau = \frac{H}{A_w}$ the average shear stress due to lateral load H; A_w – the horizontal cross section of the wall; b – the shear stress distribution factor depending on the geometry of the wall.

It is important to note that a representation of an orthotropic yield surface in terms of principal stresses only is not possible [119].

All the analytical formulae available for unreinforced masonry material can be adapted in the case of retrofitting masonry, by taking into account the experimental investigation regarding the positive effect of the reinforcing metallic plate or steel wire mesh. The replacing simple masonry with an equivalent material with improved behaviour in compression due to the confinement effect and the remove of the brittle behaviour in tension by eliminating the tension softening can be done following the experimental testing procedure presented above in this chapter, for uniaxial, biaxial and shear behaviour. This kind of approach that considers the retroffiting sheathing as an external reinforcing can really simplify the numerical effort.

5.1.2.3. Physical models for masonry infill panels – truss equivalent elements (Numerical Models)

Basic Features of Masonry Walls Modeling

In the case of masonry bracing walls, the tension field is ignored, presuming that only the compression field works, it being modeled as an equivalent diagonal strut (see Figure 5-11).

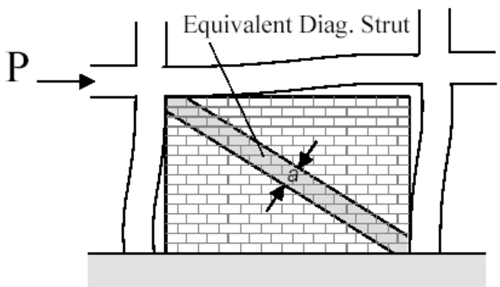


Figure 5-11 Equivalent diagonal strut [13]

Recent overviews on the seismic behaviour of infilled frames and many proposal for analytical models are available in literature, from the single-strut model, double-strut model to triple-strut model [45]. This paragraph will mentions only some of the most used ones.

It is one of the most used ways of modelling masonry bracing walls, and it consists in replacing the masonry panel through a linear brace element (Figure 5-12). Using this technique, the global analysis of the building with masonry infilled walls in both elastic and plastic domains can be performed. The capacity of the equivalent linear brace is taken as the minimum between the possible failure of the masonry wall (i.e. sliding failure, compression failure, diagonal tension failure).

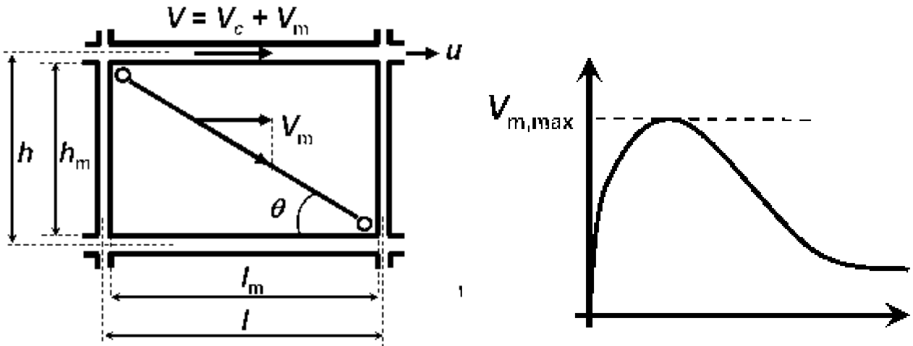


Figure 5-12 Main characteristic of the system and the constitutive law

Values for each of the resistances V_m (Stafford-Smith [198], Mainstone [126], Klingner & Bertero [111] and FEMA 306 [70]) can be established from design standards and scientific literature. For exemplification, some available formulas associated to the failure modes are given [70]:

Sliding failure $V_{m,T} = \frac{S_s l t}{0.6}$ (19)

Compression failure $V_{m,s} = \tau_0 l_m t + \mu N$, where $N = E_m l_m t \left(\frac{u}{h}\right)^2$ (20)

$\tau_0 = 0.04 f_m$

$a = 0.175(\lambda h)^{-0.4} d_m$

Diagonal tension failure $V_{m,cr} = a t f_m \cos\theta$, where $d_m = \sqrt{l_m^{\,2} + h_m^{\,2}}$ (21)

$\lambda = \left(\frac{E_m t \sin 2\theta}{4 E_c I_g h_m}\right)^{1/4}$

The following paragraphs present the most used models proposed by the scientific literature and design standards.

The model proposed by **Panagiotakos & Fardis, [176]**, like all the others models, considers the masonry panel like a strut with a specific rigidity and resistance. This model has the advantage of empirically taking into account the openings, by reducing the strut width. The constitutive law of the equivalent element is defined by the following relation for the initial rigidity k_0 and the post-yielding stiffness, and for resistance characteristics:

$$\text{Rigidity} \quad \begin{aligned} k_0 &= \frac{G_m t_m l_m}{h_m} \\ \alpha k_0 &= \frac{E_m t_m a}{d_m}\cos^2\theta \end{aligned} \tag{22}$$

$$\text{Resistence} \quad \begin{aligned} V_{m,y} &= f_{ms} t_m l_m \\ V_{m,\max} &= 1.3 \cdot V_{m,y} \end{aligned} \tag{23}$$

f_{ms} - shear strength according to diagonal compression test

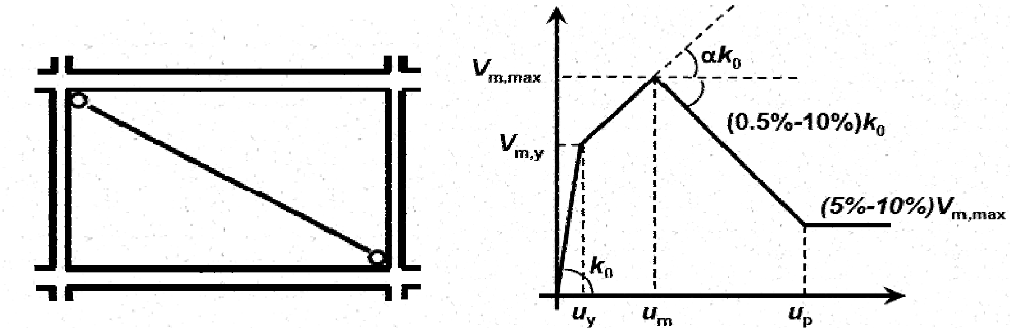


Figure 5-13 Main characteristic of the system and the constitutive law

In the model proposed by **Mostafaei & Kabeyasawa, [165],** the maximum force is chosen as the minimum, so the most probable failure mode is between sliding failure and compression failure. Also a particularity of this mode is to replace the diagonal brace with a nonlinear resort that connects the floors (see Figure 5-15).

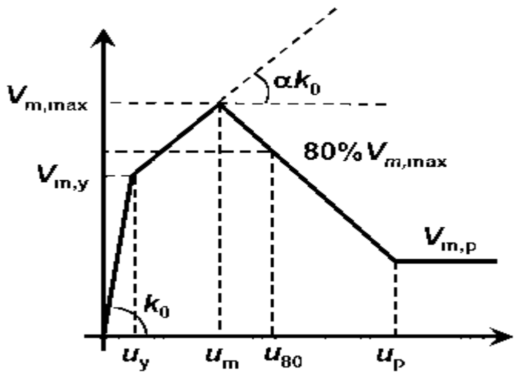


Figure 5-14 The constitutive law

$$u_m = \frac{\varepsilon_m d_w}{\cos\theta}\text{, where} \qquad \theta = arctg\left(\frac{h_w}{l_w}\right) \qquad d_w = \sqrt{l_w^{\,2} + h_w^{\,2}} \tag{24}$$

$$k_0 = 2\frac{V_{m,\max}}{u_m}; \alpha = 0.20 \rightarrow V_{m,y} = \frac{V_{m,\max} - \alpha k_0 u_m}{1-\alpha}(V_{m,\max} = 1.33 V_{m,y}) \tag{25}$$

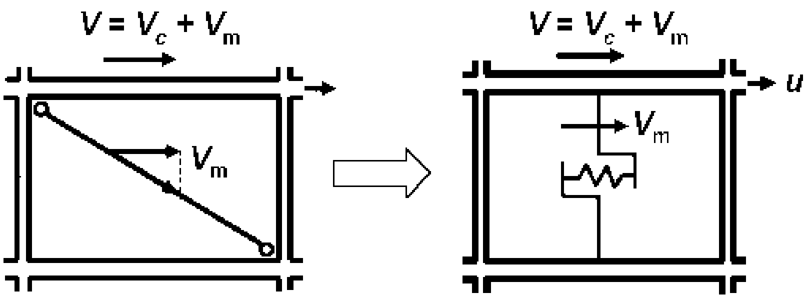


Figure 5-15 Main characteristic of the system and nonlinear spring assumption

In the American well-know standard FEMA, the **Al-Chaar, [13]** model is adopted. In a few words, one of the most important features of this model is the eccentricity (see Figure 5-16) of the equivalent brace. The width of the diagonal strut is reduced by taking into account the opening and the infill damage. The geometrical parameters are defined as follows [70]:

$$lcolumn = \frac{a}{\cos\theta column} \text{ Where } \tan\theta column = \frac{hm - \dfrac{a}{\cos\theta column}}{l} \tag{26}$$

$$a_{red} = aR_1R_2 \qquad \begin{cases} R_1 = 0.6\left(\dfrac{A_{open}}{A_{panel}}\right)^2 \\ R_2 = Table\,FEMA306 \end{cases} \tag{27}$$

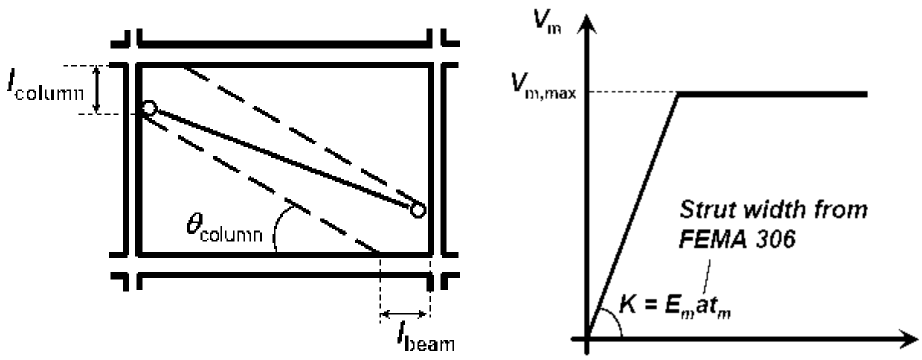


Figure 5-16 Main characteristics of the system and strut behaviour curve

The presented evaluation procedures are applicable to all building structures that have been constructed with RC frames and walls that consist of infill panels constructed of solid clay brick, concrete block, and hollow clay tile masonry. In the case of old building structures, the evaluation must be changed.

These assumptions are done in order to avoid dealing with the complicate behaviour of masonry walls and are covering the life safety requests from the codes. The numerical analyses become much easier.

This kind of supposition allow the user to perform both elastic and plastic analyzes, but without observing the state of stress distribution inside wall element or local damages, being more appropriate for global structural analysis. For monitored the local behaviour and state of stress distribution are needed planar elements and advanced numerical models and tools.

In the case of retrofitting masonry panels, the same procedure with an equivalent strut on compression and in tension can be easlly applied by introducing the experimental behaviour of the reinforcing panel. During the experimental tests carried out, the recorded data from diagonal displacement transducers can be used in order to determine the constitutive law. The experimentally obtained curves (see Figure 5-17) can be fitted and adapted for any of the models presented above, with the only modification that there is the possibility to introduce the equivalent bracing both in compression and in tension. A new constitutive law for the equivalent struts with an improvement of strength and deformation capacity will be used, either tri-linearly as presented in Chapter 4.6 or multi-linearly behaviour. This approach is to be used mainly in the case of infill frames.

From experimental results can be determinated in the cases of cyclic behaviour the main feautures of the compressive behaviour and modeled adoping the hysteresis rule proposed by Crisafulli [46] in order to takes into account the nonlinear response, including the pinching behaviour.

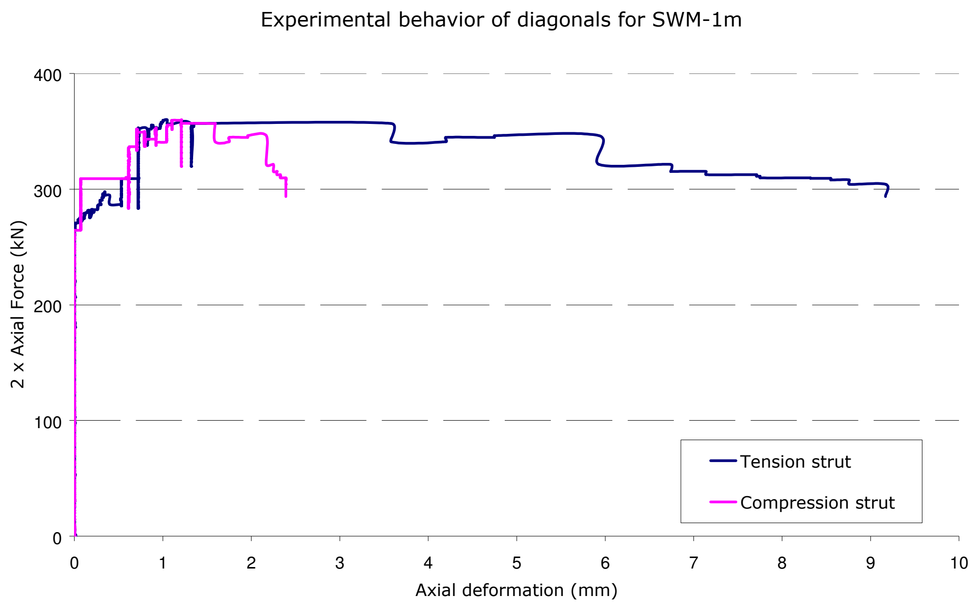


Figure 5-17 Experimental recorded curves for SWM-1m of diagonal displacement transducers

5.1.2.4. Modeling strategy for masonry façades

In literature are proposed some simple approaches to quickly asses capacity of the masonry structures. It is assumed that the most vulnerable wall plane corresponds to a façade, which can be schematized considering only the weak parts (i.e. spandrel and pier) as macroelements [29] or beam elements [123] joined by rigid elements.

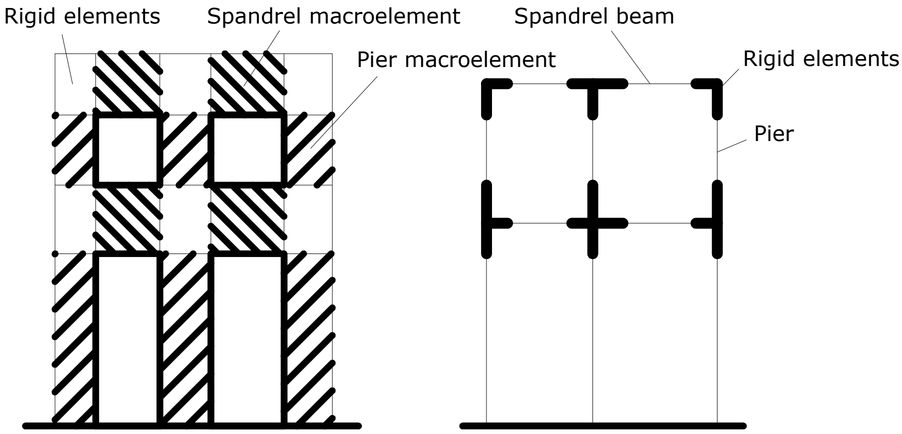


Figure 5-18 Façades models using macro-elements or a frame model [115]

5.1.3. Limit analysis for masonry buildings

Moreover, it is worth mentioning a very practical method to establish the masonry building capacity, namely the "limit analysis". Ignoring the fact that only the ultimate capacity of the structure may be obtained the easy practice of this method makes it a very attractive approach.

The modeling of masonry structures can be divided in two types: local and global modeling. Local models are expressed in terms of continuum mechanics quantities (stress and strain), whereas global models involve generalized quantities, forces and displacements. In both cases, it is difficult to obtain good results for the local analysis, due to the high scattering of material mechanical properties, and for the global analysis the empirical laws for structural elements (wall, columns, etc) are sometimes inefficient [83].

A very simple approach is the modelling of masonry buildings by rigid blocks. The use of rigid blocks theory in order to determine the limit state of masonry buildings may become a powerful tool for small and medium size historical buildings in the engineering practice. This approach avoids the use of sophisticated and time-consuming nonlinear finite element techniques. The applicability of this theory to masonry structures modeled as assemblages of rigid blocks interacting through joints depends on some hypotheses, confirmed by in-site observations and experimental results [83] [170]:

- Failure, quantify trougth limit load, occurs at small displacements, so the linear theory can be used;
- Masonry has no tensile strength, therefore tensile forces cannot be transmitted trough a mass of masonry;
- The compression and shear failures at the joints are perfectly plastic;
- Hinging failure at joint does not consider the effects of local crushing supposing that the masonry pose an infinite compressive strenght.

The method is based on the observations about the in-site formation of rigid blocks and considers the all types of collapse mechanisms, also determining the minimum collapse load for these mechanisms.

For masonry buildings it is very helpful when predicting the ultimate load and failure mode in case of an out-of-plane mechanism but also can be applied models for in-plane behaviour of walls (piers and spandrel). This models for in-plane failure will be shortly summarized in this chapter. In case of buildings, the structure is simplfied as an assemblage of the composing walls and the global behaviour will depend on the composing walls behaviour.

For in-plane behaviour the walls or façades are divided in horizontal and vertical strips depending on the openings (doors or windows) geometry. For the purpose of structural analysis, the masonry walls will be modeled through macro-panels (see Figure 5-19).

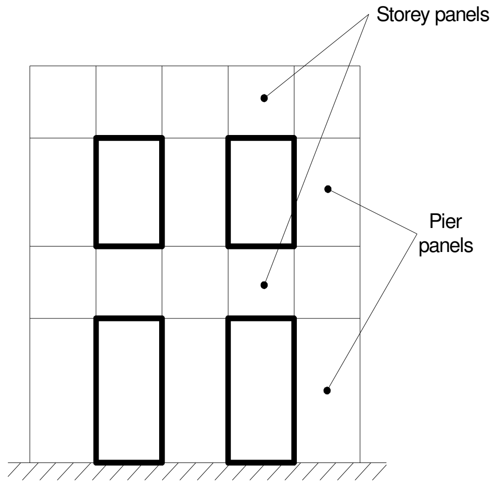


Figure 5-19 Dividing in macro-panels (pier and spandrel panels) a masonry façade [36]

The considered failure modes of a panel by using macro-panels approach are shown in Figure 5-20 [83]:

- sliding failure mode (a);
- shear failure mode (b);
- overturning failure mode (c).

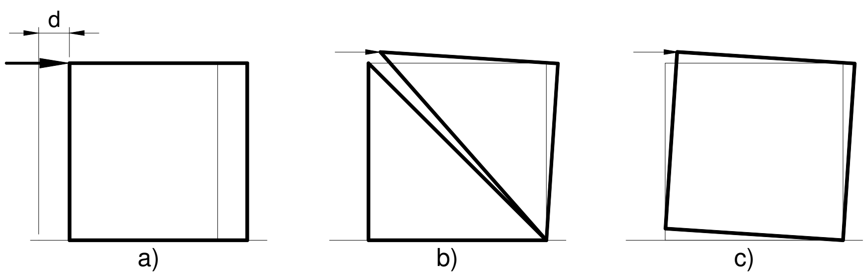


Figure 5-20 Typical in plane failure modes for a macro-panel

This approach is appropriate in order to evaluate the collapse force for unretrofitted buildings. In the case of retrofitted buildings, more sophisticated models are needed which take into account the tensile resisteance etc., and the simplicity of the method is lost.

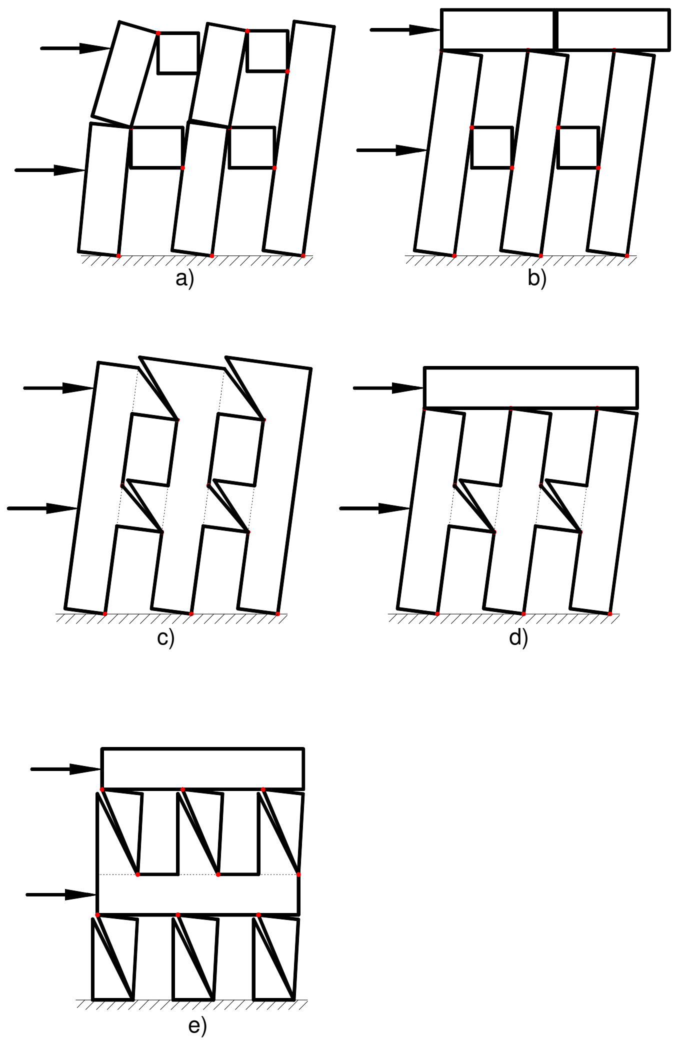


Figure 5-21 Typical in-plane failure modes for a masonry façade

5.1.4. Design assisted by testing

In some cases there are no analytical calculation procedures and numerical simulation is either difficult due to the scattering of real material properties or does not offer accurate results. Experimental tests can solve the problem. This kind of approach is based on the experimental determination of a characteristic strength of the shear wall R_k. This strength R_k is used further in order to evaluate the necessary length of the walls on a direction "i" and at storey "j" to resist the corresponding seismic shear force.

The principle of the method is presented bellow (Table 5-4):

Table 5-4 Principle of the method

$E_{s,i,j} < R_{s,i,j}$ $R_{s,i,j} = R_k \cdot L_{i,j}$	(28)	$E_{s,i,j}$- total shear force induced by seismic action in "i" direction and "j" storey; $R_{s,i,j}$- total shear wall resistance in "i" direction and "j" storey; R_k- characteristic strength of shear wall experimental determined; $L_{i,j}$- length of shear wall in "i" direction and "j" storey;

The method is applicable both for pure masonry wall and for strengthened walls (FRP, SSP – Steel Shear Plate, ASP – Aluminum Shear Plate, SWM – Steel Wire Mesh).

5.1.5. Advanced modeling of masonry

During the last forty years an enormous growth in the development of numerical tools for structural analysis has been achieved. Historical structures are particularly difficult to be analyzed due to the lack of data. Nevertheless, significant information can be obtained from numerical analysis.

Nowadays, the finite element method is usually adopted in order to achieve sophisticated simulations of the structural behaviour. A mathematical description of the material behaviour, which yields the relation between the stress and strain tensor in a material point of the body, is necessary for this purpose. This mathematical description is commonly named a constitutive model and an important objective of today's research is to obtain robust numerical tools, capable of predicting the behaviour of the structure from the elastic domain until total failure, due to excessive cracking and rigidity degradation.

5.1.5.1. Continuous modeling of masonry

The first step toward carrying out such analyses is to develop adequate constitutive models. In the case of masonry, when using the continuum model approach, three levels of approximation might be applied: micro-models, simplified or detailed, and macro-models [190](Figure 5-22):

• ***Micro-modeling*** – when units are represented by continuum elements whereas the behaviour of the mortar joints and unit-mortar interface is lumped in discontinuous or interface elements. A complete micro-model must include all the failure mechanisms of masonry, namely, cracking of joints, sliding over one head or bed joint, cracking of the units and crushing of masonry.

In the micro-model, each component of masonry – unit, mortar (simplified), and unit/mortar joint (detailed) – must be represented by with different finite elements. The employment of a micro-model to analyze an entire building becomes prohibitive, since it would result in a large number of finite elements, and consequently require a lot of computer resources to run the analyses.

Two approaches can be used: the first one is the simplified or layer model, without taking into account the interface (friction law) between brick unit elements and mortar elements (Figure 5-22a), and the second one detailed or interface model, by introducing a normal and tangential contact surface instead of mortar layers (Figure 5-22b).

These kinds of detailed and simplified micro-models have very accurate results provided that there are suitable input data. This type of analysis is the most advanced level of numerical simulation for masonry elements. It is very appropriate for simulating out-of-plane behaviour of masonry, but for in-plane behaviour this type of approach is not justified due to the high complexity compared to similar results as in easier approaches.

However, if there is a high interest in observing local behaviour and interaction with other elements or material this technique may be the only one that leads to coherent results.

• ***Macro-modeling*** – use an anisotropic continuum model that establishes the relation between average stresses and average strains in masonry, considering composite masonry as a homogeneous material

Units and joints are not represented anymore and the geometry of masonry constituents (units and joints) is lost (Figure 5-22c). An adequate macro-model must include anisotropic elastic and inelastic behaviour.

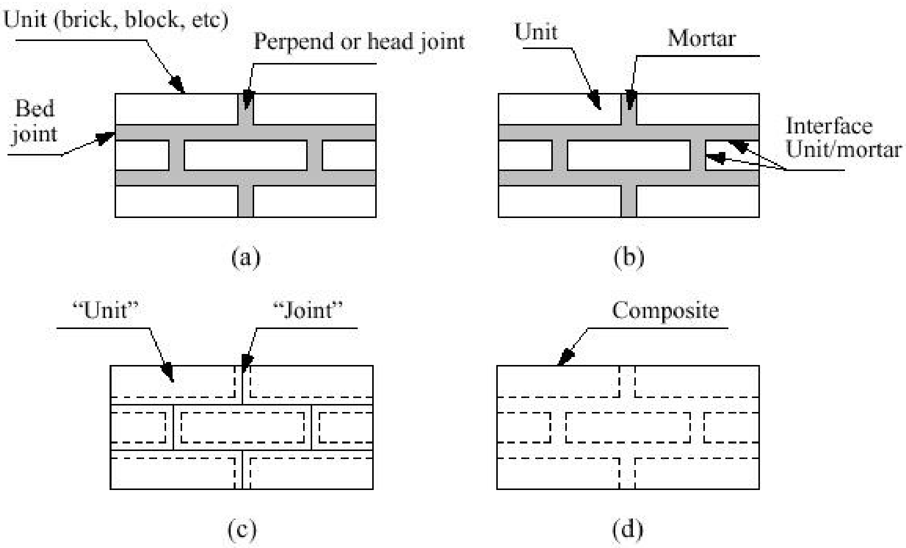
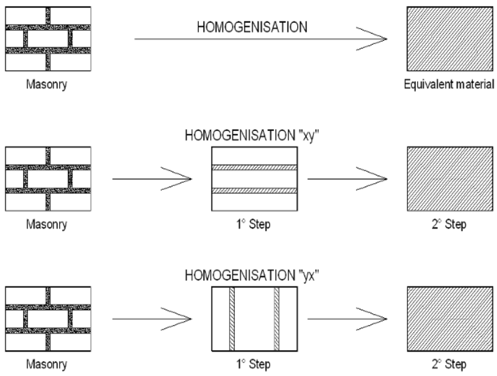


Figure 5-22 Advanced modeling approach (a) masonry sample; (b) detailed micro-modeling; (c) simplified micro-modeling; (d) macromodeling [119]

This type of analysis is the most suitable form the point of view of balance between involved time and accuracy of the results. Anyway, macro-modeling require an extra process, homogenization introduced by Salamon [191]. Homogenization of masonry is a step that has been widely treated in articles proposing complicated energy and deformation compatibility equations. Even so, the obtained results must be seriously calibrated after this homogenization, in order to obtain a good correlation with the experimental tests.

The homogenization process, proposed by [177] (Figure 5-23), in two steps, results in an elastic orthotropic material representing the anisotropic behaviour of masonry.



$$[E]^{-1} = \begin{bmatrix} \frac{1}{E_x} & -\frac{\nu_{xy}}{E_x} & -\frac{\nu_{xz}}{E_x} & 0 & 0 & 0 \\ -\frac{\nu_{yx}}{E_y} & \frac{1}{E_y} & -\frac{\nu_{yz}}{E_y} & 0 & 0 & 0 \\ -\frac{\nu_{zx}}{E_z} & -\frac{\nu_{zy}}{E_z} & \frac{1}{E_z} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{xy}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{yz}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{xz}} \end{bmatrix}$$

Figure 5-23 Homogenization steps [177]

The homogenization process described previously has many weak points as:

- it is appropriate only in the elastic range,
- it does not take into account the real pattern of the masonry wall, the results being the same for the next figure (Figure 5-24):

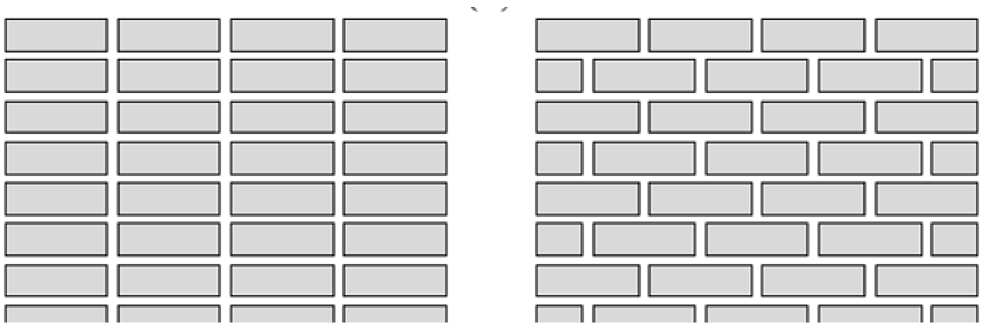


Figure 5-24 Pattern of masonry wall (a) stack bond; (b) stretcher bond. [120]

Other methods of homogenization are presented below [221] (Table 5-5)

Table 5-5 Homogenization methods (example) [221]

Methods of Homogenization	E_1 (MPa)	E_2 (MPa)	ν_{12}	G_{12} (MPa)
FEM, Stack bond (Anthoine, 1995)	8530	6790	0.196	2580
FEM, Running bond (Anthoine, 1995)	8620	6770	0.200	2620
Periodic Model, Stack bond	8568	6850	0.191	2594
Periodic Model, Running bond	8574	6809	0.197	2620
Multilayer Method (Pande *et al.* 1989)	8525	6906	0.208	2569
Two-step Method (Pietruszczak & Niu 1992)	9187	6588	0.215	2658
Elliptical Cylinder Model (Bati *et al.*, 1999)	7784	6315	0.247	2556

The most modern way of homogenization is proposed by P. B. Lourenço and A. Zucchini [122] and is based on extracting a basic-cell from the element (Figure 5-25). Different authors have chosen different cells.

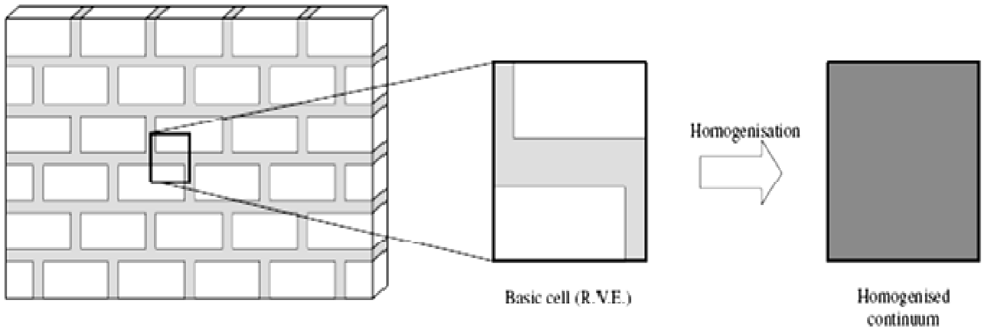


Figure 5-25 Basic cell [122]

For the purpose of understanding the internal deformational behaviour of masonry, detailed finite element calculations were carried out for different homogeneous loading conditions (Figure 5-26).

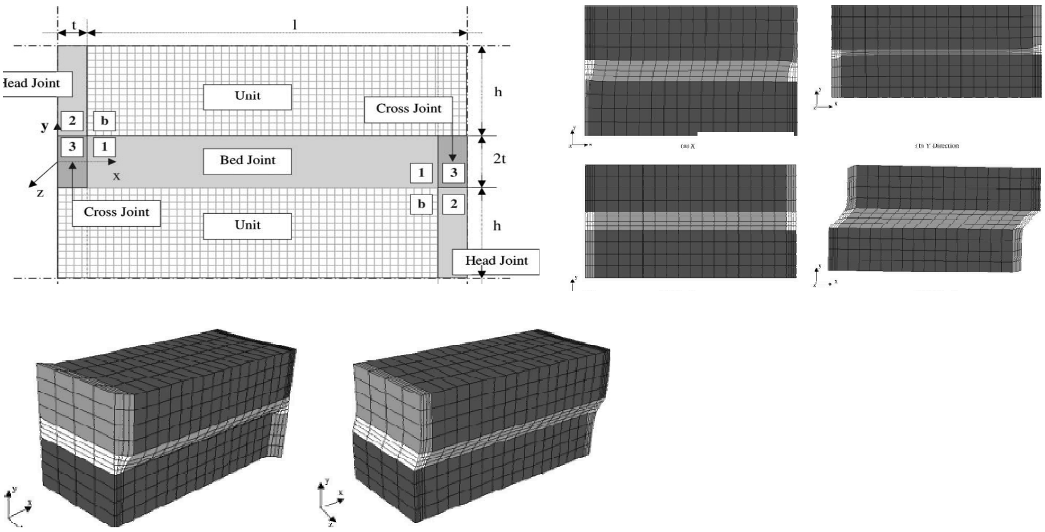


Figure 5-26 Basic cell components and behaviour [122]

By writing the simple equilibrium equation for below scheme (Figure 5-27), the elastic modulus of the "homogenous" material in the direction of loading can be obtained.

Roberto Capozucca and Fabrizio Collini [38] have developed, based on Lourenço theory, a homogenization technique for the analysis of a shear wall. They have studied a panel of small width s compared to the dimensions of the wall in the x-y plane.

The wall is considered to be stratified with the thickness of layer h_i. The continuum is considered transversally isotropic as a result of the symmetry around the vertical axis y. The stress and strain relationships for the homogeneous continuum are evaluated considering the equivalence of energy of stratified element, U_r, with the energy of homogeneous element U_0 [38].

$U_r = U_0$;

The energy of the stratified element is expressed as follows:

$$U_r = \frac{1}{2}\sum_i \int_{V_i} \delta_i^T \varepsilon_i dV \tag{29}$$

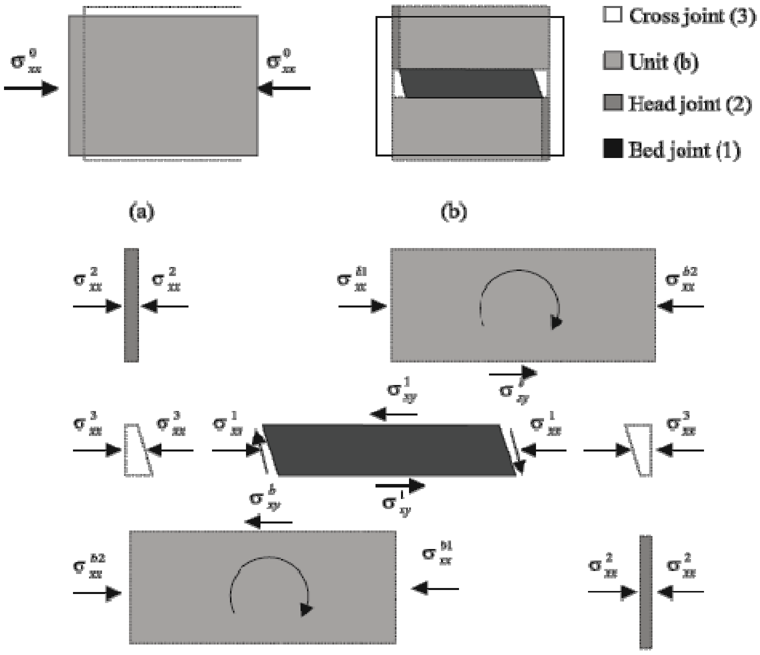
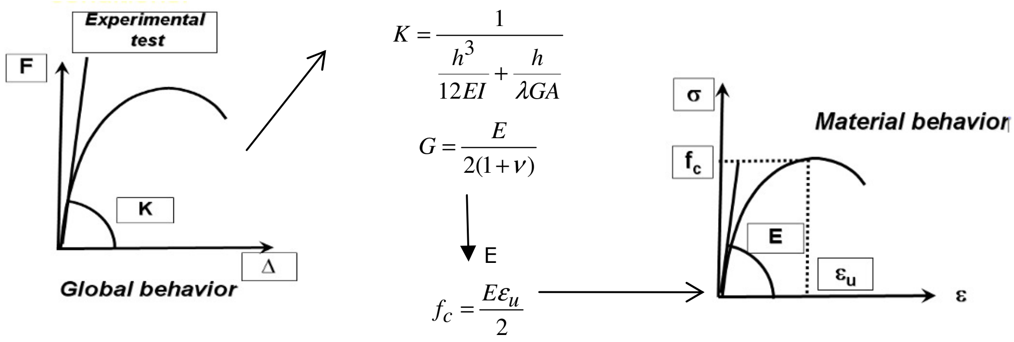


Figure 5-27 Basic cell stress conditions [38]

A more simple technique for homogenization is proposed. Starting from the experimental global behaviour we can extract the rigidity of the element and obtain the elastic modulus. After establishing an initial value for the elastic modulus and compressive ultimate stress of the material, the numerical simulation and calibration of the model can be obtained in order to achieve a good fit of the experimental results with the numerical simulation.



Appling a constitutive law

5.1.5.2. Discontinuous modeling of masonry [211]

Recently a considerable attention has also been given to rational assessment methodologies, to be directly consistent with the discontinuous nature of structural masonry.

The discontinuities in continuous systems are in fact interfaces between dissimilar materials and joints or fractures in the material. A survey of the literature on finite element modeling of cracks and joints shows that two main approaches are common for a representative analysis: the *discrete crack* and the *smeared crack* approach and the use of *joint* or *interface* elements.

Discrete crack approach explicitly represents the crack as a separation of nodes (Figure 5-28). When the stress or strain at a node, or the average in adjacent elements, exceeds a given value, the node is redefined as two nodes and the elements on either side are allowed to separate. This method produces a realistic representation of the opening crack, but a coarse meshing in the finite element model may result in the misrepresentation of the propagating crack. Other disadvantage is the changing of the formulation of the finite element model that increases the number of equations to be solved and extend the bandwidth of the stiffness matrix.

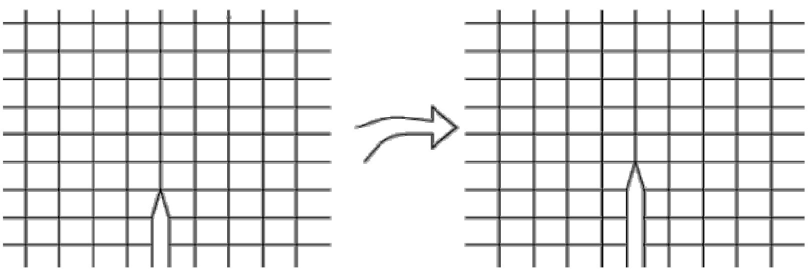


Figure 5-28 Discrete crack

In the **Smeared crack approach**, cracks and joints are modeled in an average sense by an appropriate modification of the material properties at the integration points of regular finite elements.

Smeared cracks are convenient when the crack orientations are not known beforehand, because the formation of a crack involves no remeshing or new degrees of freedom. However, they have only limited ability to model sharp discontinuities and represent the topology or material behaviour in the vicinity of the crack.

The *smeared crack* concept, based upon strain decomposition and first developed for use in concrete structures, has also been extended to the analysis of masonry elements. The method is attractive if global analysis of large-scale masonry structures is required. It does not make a distinction between individual bricks and joints, but treats masonry as an anisotropic composite such that joints and cracks are smeared out. An inherent limitation of the smeared crack approach is that discrete cracks are smeared out over an entire element and the crack opening is modeled by the continuous displacement approximation functions of the conventional finite element approach (Figure 5-29). In view of this limitation, as well as other problems such as mesh-dependency due to tensile and compressive softening and difficulties of model calibration, smeared crack models should only be used with caution for the analysis of discontinuous structures.

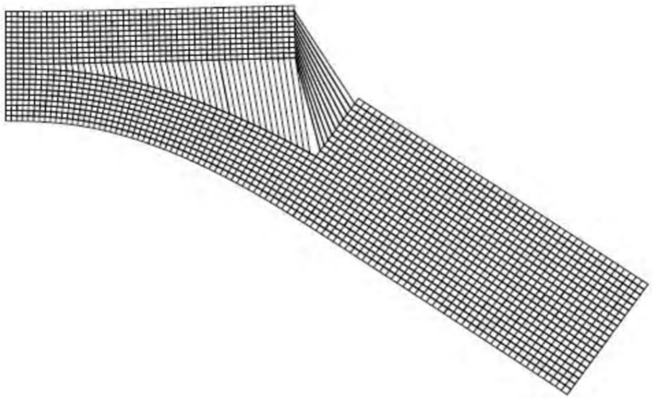


Figure 5-29 Smeared crack [9]

The Interface smeared crack approach combines the advantages of the discrete and smeared approaches described above.

The model treats cracks discretely like joint elements, but, like smeared crack elements, it does not introduce additional degrees of freedom. Cracking is limited to element boundaries and, if the crack opening criterion is met at a boundary node, then the local element displacements are altered until stresses perpendicular to the interface are brought as close as possible to zero.

Other methods of approach use the *crack band theory* to model the tensile behaviour of concrete. The fracture of concrete is represented as a band of smeared cracks over a *crack band* of a certain width. Micro-cracking in the band is identified with the phenomenon of strain softening, which is represented by a stress-strain relationship that preserves the fracture energy of the material.

For modeling the discontinuities present in a system *the method of constraints* can be also used. According to this approach, interface discontinuities are represented by a sequence of double nodes, one on each side of the interface. The interconnection between the double nodes is controlled soas to simulate the physical behaviour of the interface, and the desired solution is obtained by modifying the global stiffness equations in such a manner that all the interface conditions, such as compatibility and friction law, should be satisfied.

All the *crack* models reviewed above have only limited ability to model sharp discontinuities present in many structural systems. **Joint elements** are more appropriate for modeling the opening and closing of discrete cracks and joints and have been used in numerous applications. For the efficient non-linear analysis of masonry, it is necessary to consider relative slip, debonding and cycles of closing and opening of the interfaces.

For wall thickness t_w and mortar joint thickness t_m, the normal and shear stiffness required to define the material property matrix can be represented by the following expressions [172]

$$K_{sx} = K_{sy} = G\frac{t_w}{t_m} \qquad K_{nz} = K_{sy} = E\frac{t_w}{t_m} \tag{30}$$

In which E and G are the instantaneous tangent elastic and shear module at the particular value of normal and shear stress considered. The non-linear behaviour of the joints can therefore be treated by assigning the joint properties corresponding to the level of stress obtained from the last load step in a step-by-step loading analysis procedure.

The failure criteria of a joint depend mainly on the relative magnitudes of the normal and shear stresses present in the joint. The relationship between the normal stress in a joint and its ultimate shear strength can be obtained from tests on masonry prisms with the load inclined to the bed joints.

Applications using discontinuum models may be carried out using finite elements, discrete elements or limit analysis. The salient features of this types of discrete elements may by summarized [118]:

- Rigid or deformable (combined with the FE method) blocks;
- Connection between vertexes and sides / faces;
- Interpenetration possible, integration of the equation of motion;
- Real damping coefficient (dynamic problem) or artificially high damping (static solution)

5.1.6. Analysis types and performance criteria

According to Eurocode 6 [64] linear analysis, there are recommended performance criteria in terms of limit states:

- Ultimate limit state associated with collapse or with other forms of structural failure,
    - loss of equilibrium of the structure or any part of it, considered as a rigid body,
    - failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.
- Serviceability limit states correspond to states beyond which specified service criteria are no longer met.
    - deformations or deflections which affect the appearance or effective use of the structure (including the malfunction of machines or services) or cause damage to finishes or non-structural elements,
    - vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.

5.2. FE ANALYSIS

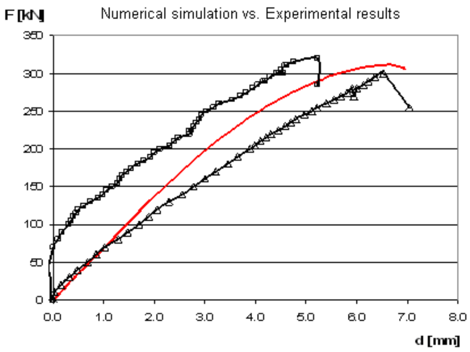
5.2.1. Preliminary FE Models

This paragraph describes the numerical model for masonry panels strengthened by steel plates connected with chemical anchors.

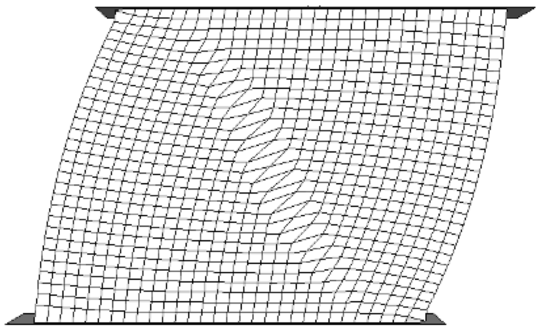
Within the framework of the research activities there were studied different material models for masonry available in ABAQUS Library [9] using the macro-model approach.

In the first phase, before the experimental tests, the numerical program was dedicated to predict the behaviour of the system in order to prepare the experimental program and to choose the optimal parameters for the retrofitting systems. This phase was divided into two steps: the first one was focused on to calibrating a masonry wall numerical model (based on experimental tests previously carried out previous at The Department of Civil Engineering from "Politehnica" University of Timisoara) and the second one on to describing beforehand the behaviour of the retrofitted elements.

In a first attempt, a "*concrete smeared cracking*" model for the masonry was applied, based on two parameters for the elastic range (e.g. Elastic Modulus and Poisson Ratio), and seven parameters in the plastic range (e.g. failure ratios for describing biaxial behaviour and failure surface, tension stiffening and shear retention (reduction in the shear modulus as a function of the opening strain across the crack) [9]. Using calibrated parameters, a very good behaviour was obtained, in terms of global performance indicators and failure mode for unreinforced masonry panel (see Figure 5-30).



a) Global behaviour



b) Deformed shape of the panel

Figure 5-30. Numerical simulation vs. Experimental results

In addition, a reduced numerical model of the retrofitted system applied on one side through chemical anchors (only with 9 connectors) was numerically analyzed. The unreinforced masonry panel (URM) was calibrated using the experimental results available in the literature. The results for masonry panel and sheathing plate are showed in Figure 5-31 [37].

These numerical simulations led to a series of conclusions. Both the resistance and the ultimate displacement were improved by an increase in the range of 30-50%. The failure mode of the retrofitted masonry wall was changed from the diagonal cracking failure (shear at principal tension stress), of the unreinforced model to the tension failure and compression crushing of the corners in the retrofitted model (eccentrically compression). The thickness of the steel plate is not so relevant in the considered range [37].

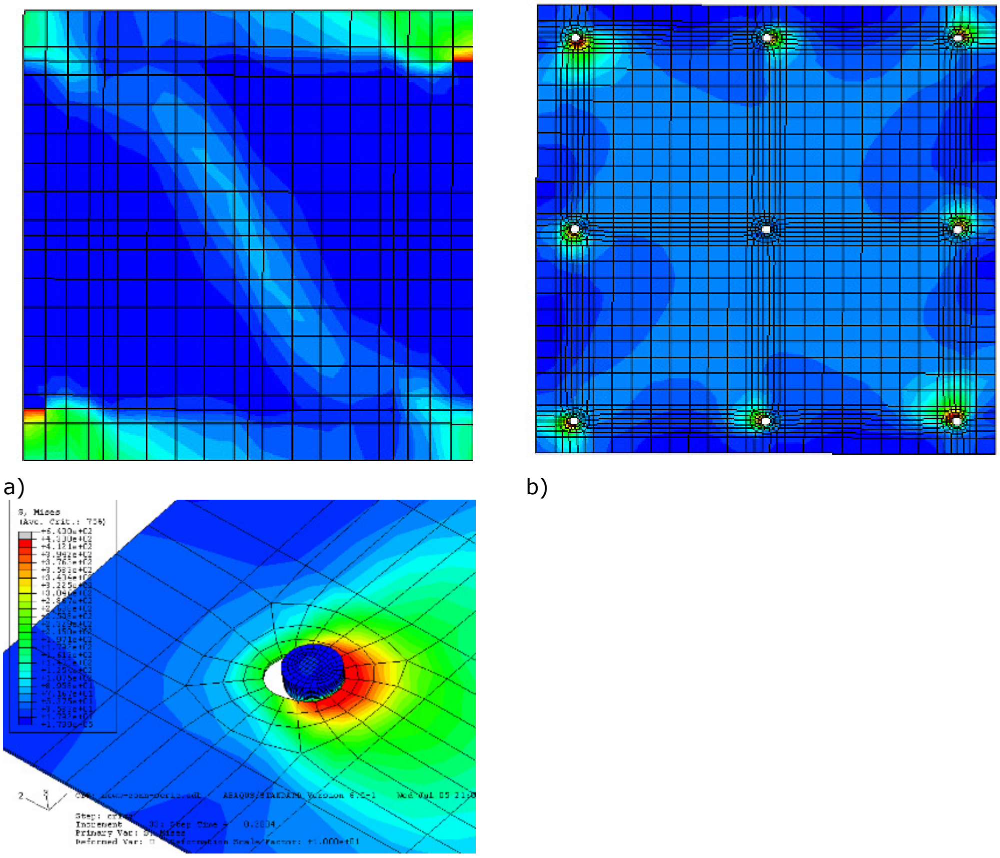


Figure 5-31. State of stress in (a) the masonry panel, (b) steel plates and (c) around steel connector

In the case of strengthened masonry panel, pre-experimental numerical simulation have been compared to the experimental results, the "smeared crack" model showing its limitations. The model is not capable to describe large ultimate displacements. Therefore, even the test showed comparatively to unstrengthened masonry an increase of the ultimate displacement of 3-5 times more, in the case of retrofitted elements, the contribution of confining effect by sheathing plates cannot be replicated.

So, new material models to describe better the masonry shear walls behaviour are needed. *Brittle cracking* and *Concrete Damage Plasticity*, models available in ABAQUS library [9] were studied. The common feature of the above material models is the possibility to simulate the brittle failure of the material by a smeared cracking approach (relaxing of the internal stiffness of the material). One of the main advantages of these models is the possibility to use a dynamic formulation of equilibrium that offers more quicker results and a more robust numerical tool.

Brittle cracking model is suitable for evaluating of the brittle behaviour of materials, i.e. masonry. The failure criterion is introduced only in the tension range. The model neglects any compressive ultimate stress/deformation considering infinite compressive strength [9]. A quasi-static analysis with an explicit solution has been conducted and the results are plotted in Figure 5-32.

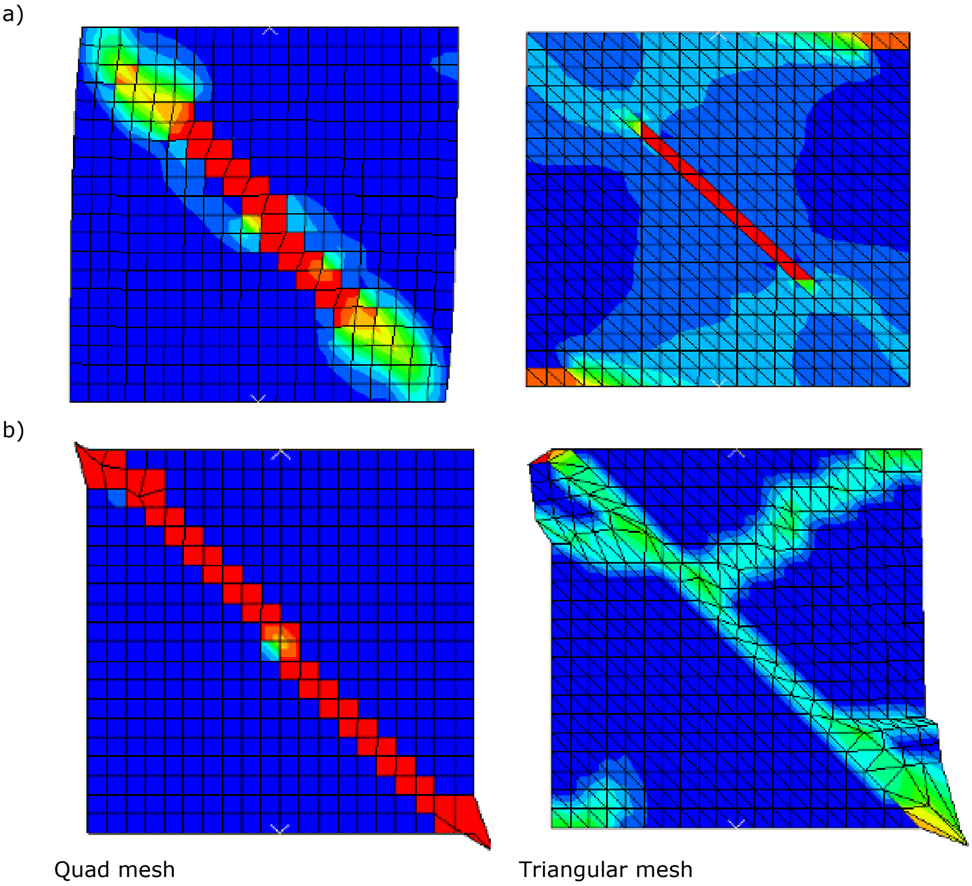


Figure 5-32. Masonry panel at peak force (a) and ultimate displacement (b)

The model exhibits a very large displacement and from this point of view it can be suitable for use in the case of retrofitted models where large displacement and important ductility are expected to develop. Moreover the failure mode of the numerical masonry model, i.e. diagonal tensile crack, is as observed on the experimental tests.

Unfortunately, mesh instability develops and a very high energy unbalance appears in the post cracking state of the model for both types of mesh used, quad and triangular.

Concrete damage plasticity is oriented for the analyses of concrete elements. Even so, the brittle behaviour of masonry, together with the cracks development, can be simulated with good accuracy. The material model provides both crushing (in compression) and cracking (in tension) failure (see Figure 5-33), but doesn't introduce shear retention , assuming that the shear response is unaffected by cracking [9]. This assumption can be reasonable, the overall response, in most cases, is not strongly dependent on the amount of shear retention.

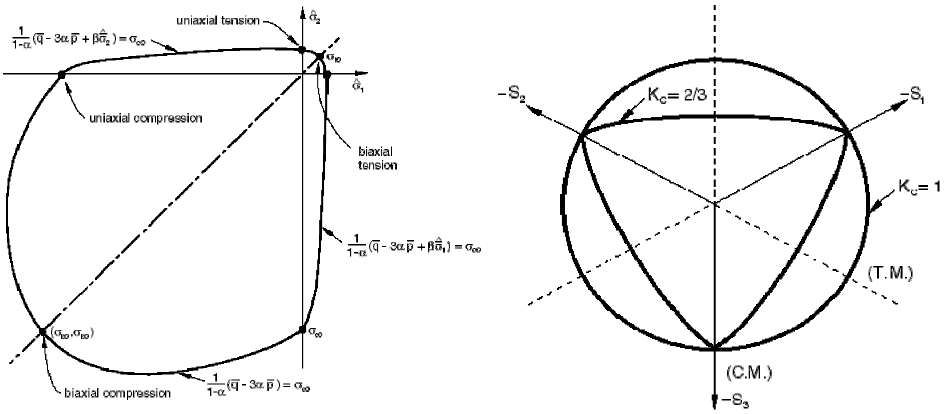


Figure 5-33. Yield surface in plane stress and yield surface in deviatoric plane [9]

The performed numerical analyses has been carried out by introducing a parabolic-rectangular uni-axial constitutive law in compression (Figure 5-34), similar with the EC specification [64] (see Table 5-2), that has been calibrated based on the results of the experimental campaign on masonry unreinforced shear-walls. A 7.5 N/mm^2 has been considered as ultimate compression strength at 0.18% strain.

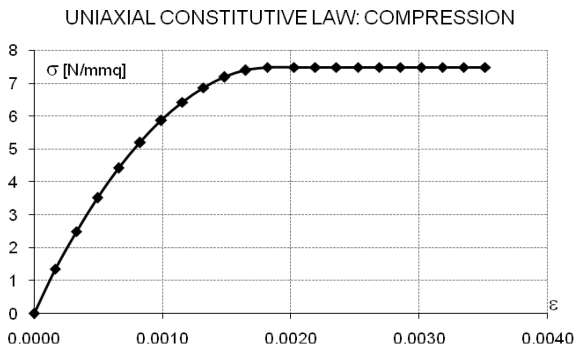


Figure 5-34. Assumed masonry constitutive law in compression

In the tensile range, a uni-axial softening is assumed to reproduce cracking failure, defining a stress-displacement relationship. It was considered that at 0.65 mm crack opening the tensile resistance is completely lost. This relationship is shown in Figure 5-35.

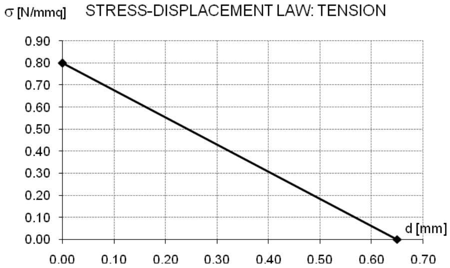
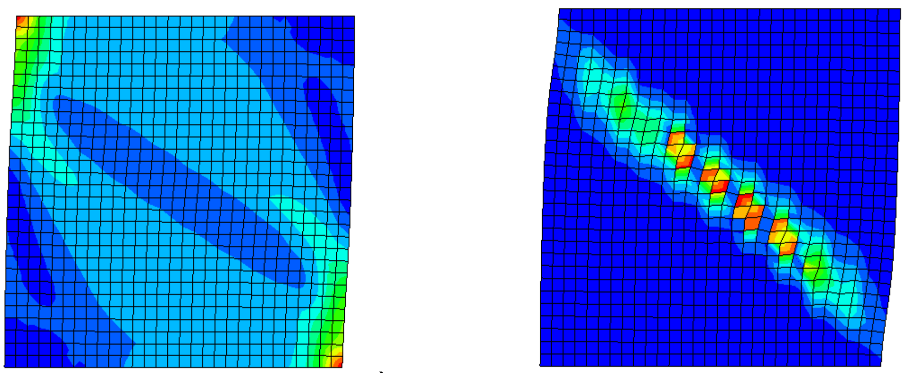


Figure 5-35. Assumed masonry constitutive law in tension. Post-failure stress-displacement curve

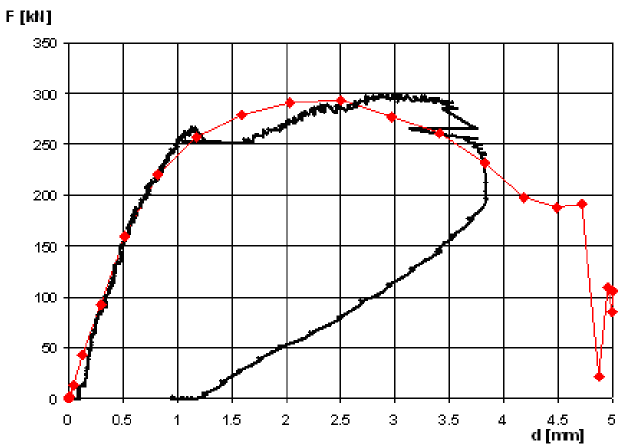
A quasi-static analysis with an explicit solution has been performed and results are presented in Figure 5-37.

A quasi-static analysis with an explicit solution has been performed and the salient results are presented in Figure 5-36.



a) b)

Figure 5-36. Masonry panel Von Misses stress (a) and distribution of cracks (b) at failure



---- Experimental unreinforced model ----- Numerical model

Figure 5-37. Experimental behaviour curve of masonry and numerical simulation

The model exhibits a very large ultimate displacement and from this point of view it can be suitable for our purpose; moreover, the failure mode of the masonry wall reflects the observed real shear failure, i.e. diagonal tensile cracking. The mesh is able to conserve a good stability state and there are no significant energetic unbalances. A proper calibration starting from the experimental results was achieved, as is shown in Figure 5-37.

5.2.2. FE Model for retrofitted masonry sheathed with metallic plates

5.2.2.1. Description of the numerical model

Some simplifications have been adopted in order to reduce the amount of time of the numerical elaborations: the geometry of the model has been reduced using the symmetry of the system in respect to the mid plane. Only the half-part of the specimen has been studied by introducing proper boundary conditions (see Figure 5-38); the link between connectors and masonry was simplified in sequence of node to node internal constraints and by using gap elements. Although in reality the steel connector is infilled in the masonry wall, the implemented ABAQUS model doesn’t respect the physical situation (see Figure 5-39). This modelling strategy has been adopted in order to avoid the perforation of the masonry wall model so as not to have heavy irregular mesh elements around the holes. It is well known that extremely simple and regular meshes are required for numerical analyses on brittle materials with a smeared cracking constitutive law.

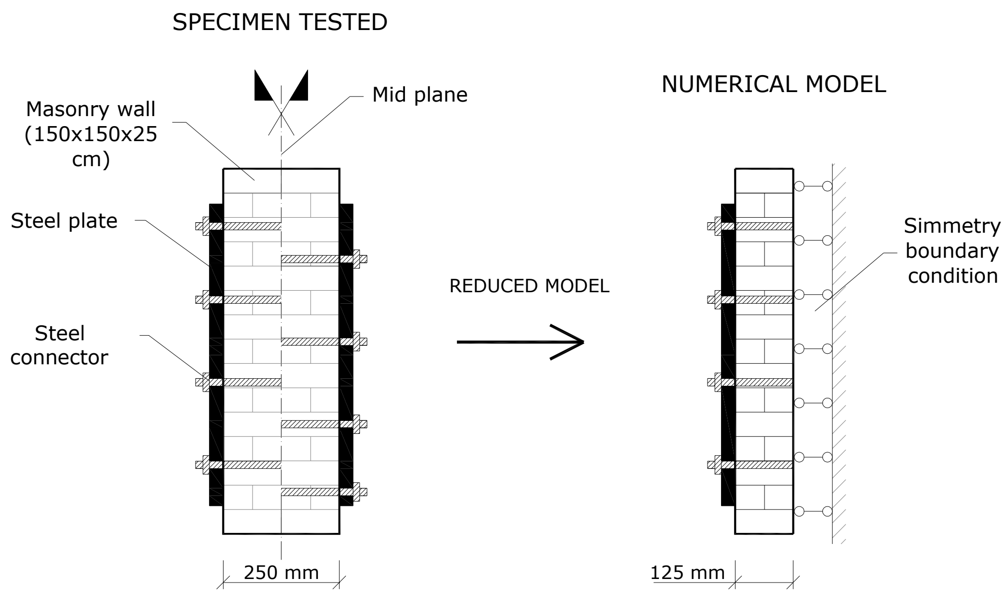


Figure 5-38. Scheme of the cross section of the reinforcing system investigated

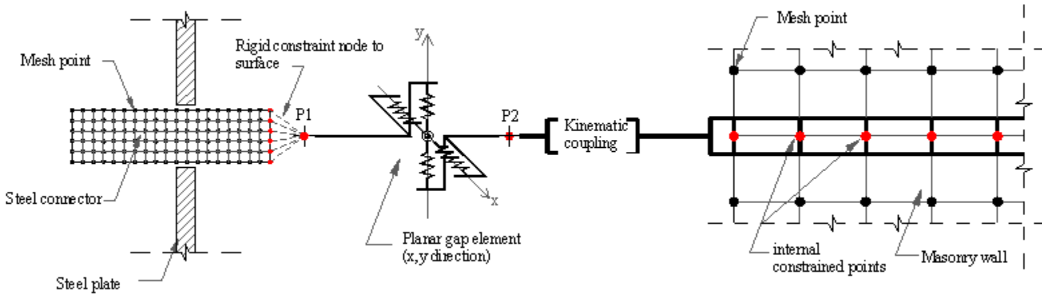
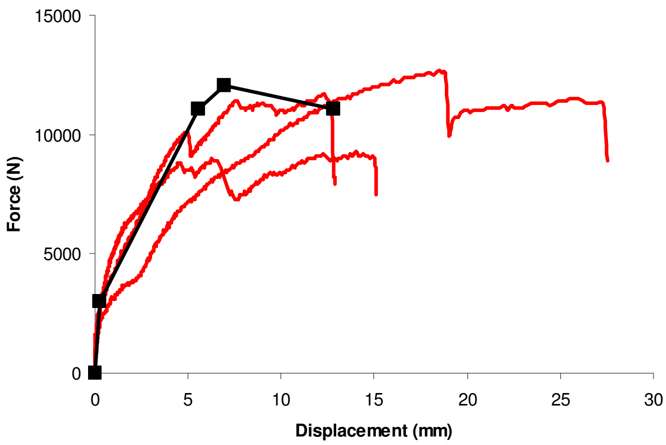


Figure 5-39. Modelling strategy for the connector

Referring to Figure 5-39, it is interesting to explain the single elements used for gluing the steel connector part to the masonry wall without any perforations. First, a reference point P1 has been defined, then a node to surface rigid constraint has been introduced between the base surface of the steel connector and P1. With this assumption the base of the connector moves as a rigid surface in 3D space presenting the same displacements assumed by P1. In other words in P1 are concentrated the resultants forces and moments obtained by the integration of the stresses on the base surface of the steel connector.

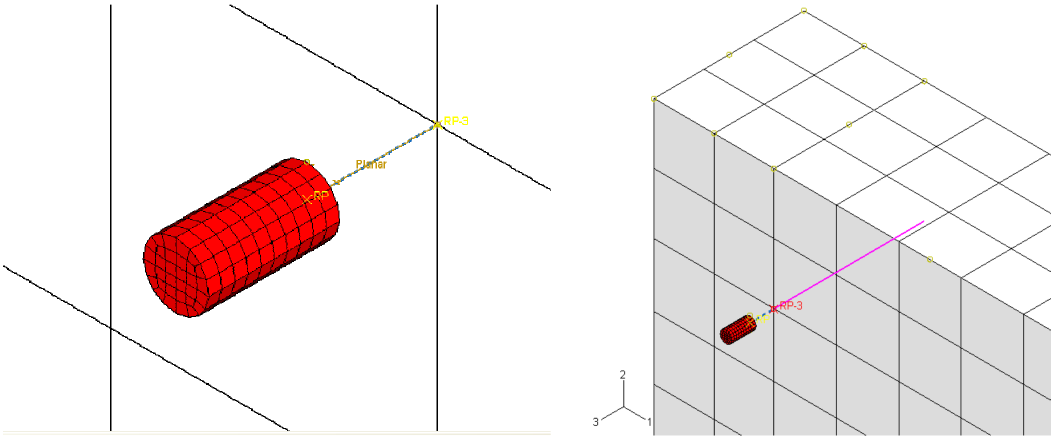
Another point P2 has been introduced and a planar gap element is defined between P1 and P2. These gap elements consist in planar springs in the plane of the masonry wall (plane x-y reported in Figure 5-39) that is able to transfer the resultant shear force acting in P1 because of the contact between the steel connector and the steel plate. The gap element concentrates all the shear deformability of the steel connector. No relative out of plane displacements between P1 and P2 can appear, thus reproducing the higher axial rigidity of the steel connector. The constitutive law of the springs introduced in the gap element has been directly derived from experimental tests made on the same steel connectors loaded in shear (see 4.5.1.1). This constitutive law is shown in Figure 5-40.



F = shear force in the plane of masonry wall at the base of the connector; d = displacement of the base of the steel connector in the plane of the masonry wall.

Figure 5-40. Constitutive laws of the gap elements

Finally, P2 has been tied with the internal points of the masonry wall by means of an internal constraint ("kinematic coupling"), thus equally distributing all the components of the forces acting in P2 over all the internal points of the masonry wall aligned with the steel connector. With this assumption the global force in P2 is transferred only to the points of the masonry that are inside the control volume of the masonry directly influenced by the presence of the steel connector. The gap element and the kinematic coupling as defined in the model are shown in Figure 5-41.



a) Planar spring
b) Kinematic coupling

Figure 5-41. Detail of the internal constraint defined in Abaqus model

This procedure has been repeated for each of the 36 steel connectors present in the retrofitted model (see Figure 5-42).

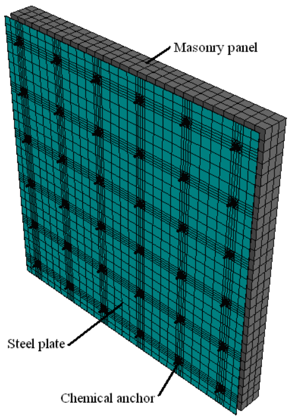


Figure 5-42. Geometry of the retrofitted model

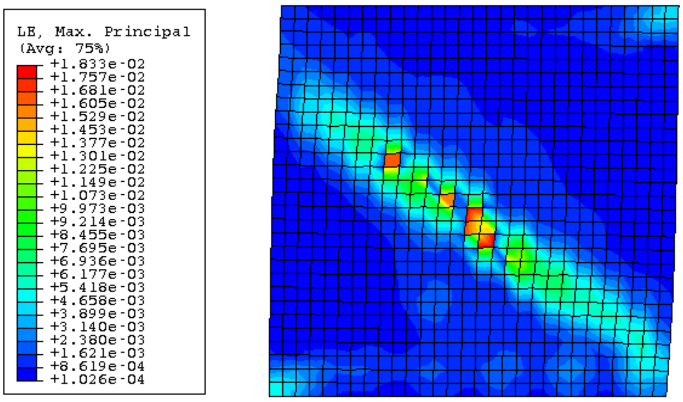


Figure 5-43. Logarithmic strain in the masonry panel at 4 mm displacement

5.2.2.2. Numerical results and conclusion

The behaviour of the retrofitted model is also described in Figure 5-43 and Figure 5-44, showing a diagonal tensile failure similar to the one experimentally observed.

As showed in the previous Figure 5-44 the steel plate works with low values of stresses except for areas around steel connectors, in which, because of the contact, yielding stress is reached.

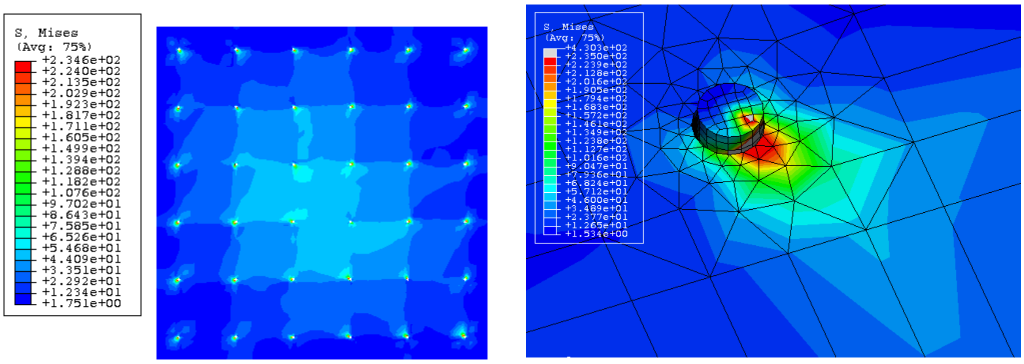


Figure 5-44. Numerical results on retrofitted model – Von Misses stress – steel plate and around steel connector

As we can see in Figure 5-45, the adopted modeling strategy to simulate the behaviour of retrofitted masonry panel has revealed the same global response observed in the experimental test also in terms of force – displacement and in terms of failure mechanism.

These numerical models available in the finite element dedicated software can be used for any type of brittle material like masonry or concrete.

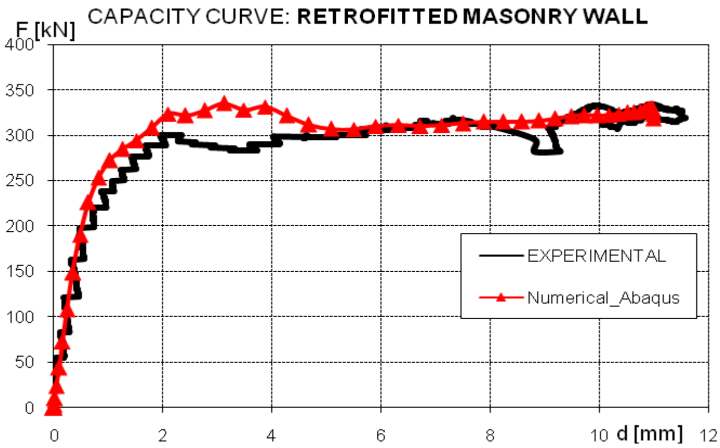


Figure 5-45 General shear-displacement behaviour of the system

Once carefully calibrated, this numerical tools may be used in order to replicate the experimental tests. The "numerical experimentation" may establish the effect of retrofitting technique on different types of masonry material and wall geometry, allowing a proper solution design.

6. PRACTICAL APPLICATION. STUDY CASE

6.1. INTRODUCTION

The proposed retrofitting technique described in this thesis [55] was proved to be suitable for a Performance Based Seismic Assessment. In order to apply this concept, acceptance criteria and numerical simulation are needed so as to sustain and extend experimental observations.

Although at the level of a single element the solution has shown good experimental results ([55], [60]), in order to validate this technique using the Decisional Matrix presented in the first part of this paper, the application of on a real masonry building is required.

Because of the very complex masonry – steel structural system, it is very difficult to apply simple assumptions and evaluate procedures concerning the retrofitting effect; thus advanced numerical models, build in agreement with all the details (e.g. masonry specific layout, connection behaviour, etc.) must be used in order to predict the real behaviour. This very detailed model, shown in the previous chapter (5.2.2.1), is almost impossible to be applied for global analysis; even if advanced tools are used supplementary simplifications must be made. The idea has arisen to find an equivalent material to replicate the behaviour of the retrofitted model. This simplification must be carefully analyzed and argued.

The advantage of such a model is the possibility to apply the nonlinear analysis and to characterize the global behaviour of the building in terms of drift ratios, which gives the possibility to use the FEMA 356 criteria for the validation and performance levels' characterization.

This chapter contains a complete performance based seismic assessment of a case study, an XX century masonry building.

6.1.1. Description of the building subjected to retrofitting

This is a general description of a masonry building, located in Toscana region, Italy and designed according only to geometrical considerations (as typical at the beginning of the XX century); this building was selected as reference benchmark structure for the performance analyses of the steel intervention techniques within the framework of STEELRETRO Project (RFSR-CT-2007-00050) [167]. For PBSA, an intensity of 0.24g of PGA and type B soil have been considered.

The reference building respects all the main features of traditional masonry buildings, ground floor plus two floors, symmetrical in plan and elevation, with small and well positioned openings, with an almost cubical shape of 15m width, length and height. The bearing wall thickness varies from 350 mm to 650 mm and is made of stone masonry.

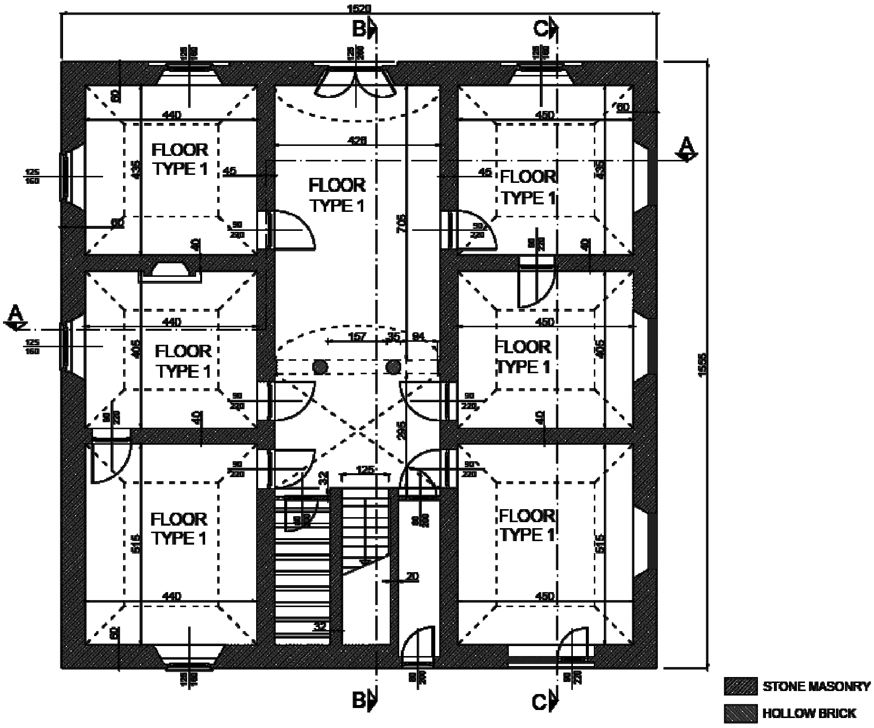


Figure 6-1. Plan view of the first floor

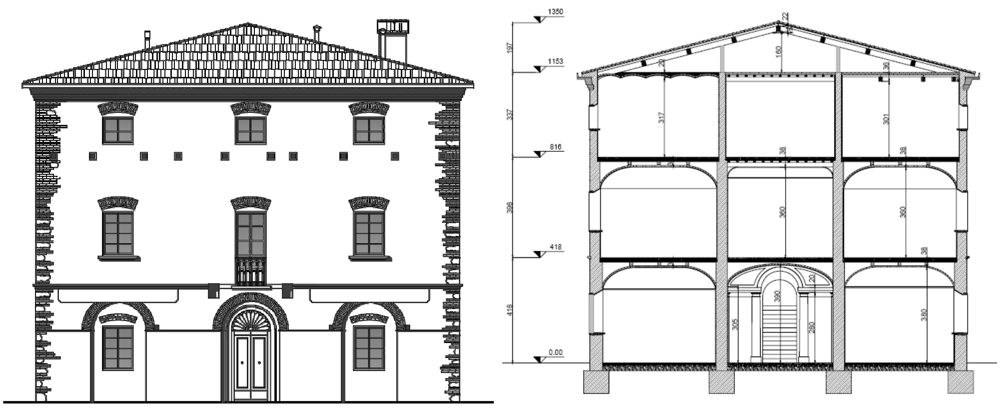


Figure 6-2. Plan view of the North façade and vertical section

The material properties adopted for the structural modelling of the masonry benchmark are drawn from literature [169].

The walls are built from stone masonry with the following mechanical characteristics: mean compressive strength f_m = 1.5 MPa, Elastic Modulus E_m = 1500 MPa and mean unit weight w = 21 kN/m^3.

In order to apply the metal sheathing retrofitting to the walls, there were assumed the measures necessary to provide the diaphragm effect of floors and roof, the integrity of the wall junctions had already been done.

6.1.2. Application of the intervention technique to the building

The building façade was reinforced on the entire height of the building as shown in Figure 6-3, and all the internal transversal shear walls from the ground floor (see Figure 6-3). Other possible location of sheathing would be at the corners of the entire ground floor. Beside structural aspects, the selection of intervention solutions must consider the costs and time, as well as the aesthetical reasons. The possibility to keep using the building even partially during intervention is also very important.

The applied techniques attempt to be minimal and to avoid affecting the internal walls, so as not to disturb the occupancy of the building.

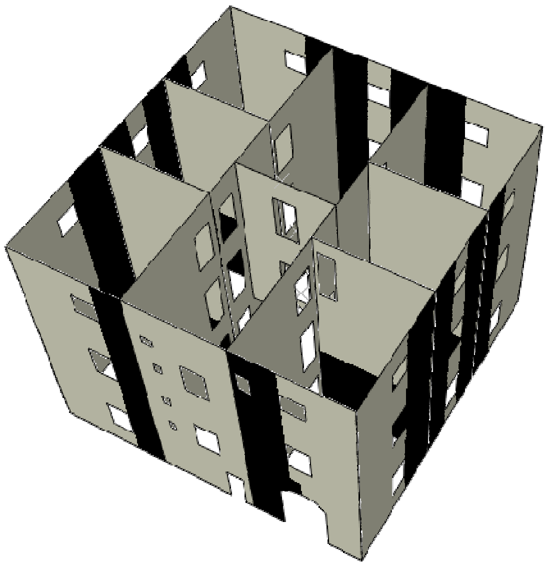


Figure 6-3. Reinforcing areas

6.2. PERFORMANCE BASED EVALUATION

6.2.1. Nonlinear model and specific acceptance criteria

A proper application of PBSA needs a reliable nonlinear analysis model in order to perform advance displacement control analysis.

The ABAQUS numerical model applied in this study was calibrated on the basis of experimental tests and is present in detail in previous chapter (5.2.2). A homogeneous macro-model with a Concrete Damage Plasticity material model [9] has been used.

The performance based approach considers the structure as an assembly of its individual components. The building performance level should be defined in relation to its element performance. The evaluation of the effects of damage on building performance must concentrate on how component properties change as a result of the damages. The response of the components is controlled by force – deformation properties (e.g. elastic stiffness, yield or cracking point, ductility, and ultimate deformation) [16][17].

Damage affects the behaviour of individual elements differently. Some exhibit ductile modes of post-elastic behaviour, maintaining strength even with large displacement. Others are brittle and loose strength abruptly after small inelastic displacement or strain. As earthquake shaking imposes these actions on the component, the latter tends to exhibit predominant modes of damage behaviour. The behaviour of a panel depends on its strength in flexure relatively to the strength in shear. Cracks and other signs of damage must be interpreted in the context of the behaviour mode of each specific component.

A complete evaluation must take into account the cracks width, location, orientation, and their number and distribution pattern. In a simple manner cracks width is commonly used to determine the damage level or performance of the wall. The performance acceptance criteria were established on the retrofitted wall panel model in terms of plastic strain at a certain performance level. A quicker assessment of the overall performance can be based on shear stress. The reinforced panel numerical model fails due to compressive load by crushing of the masonry. If the unreinforced model fails at approximately 0.15% of the plastic strain, in terms of tensile strain in shear diagonal strip the retrofitted models allow for reaching more than 3.5% strain before collapse prevention level and failure (see Figure 6-4).

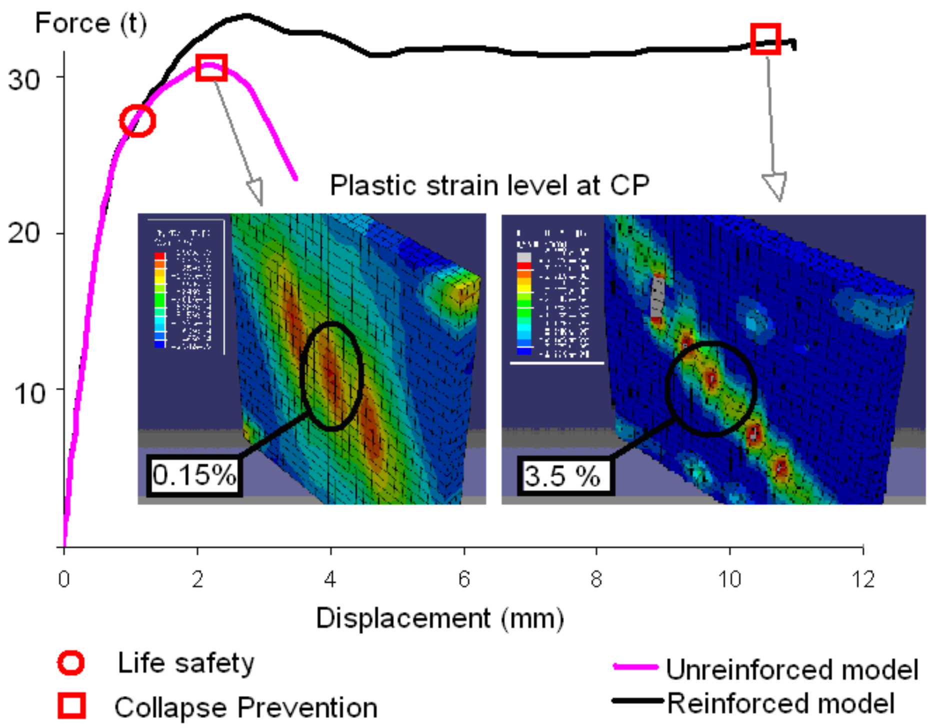


Figure 6-4. Global behaviour curves and maximum plastic strain level at failure

The global behaviour curves (see Figure 6-4) come to sustain, ones again, the possibility to enhance the deformation of the masonry wall and prove suitability to apply the performance levels presented in Figure 6-5, thus showing the benefit of the applied reinforcing.

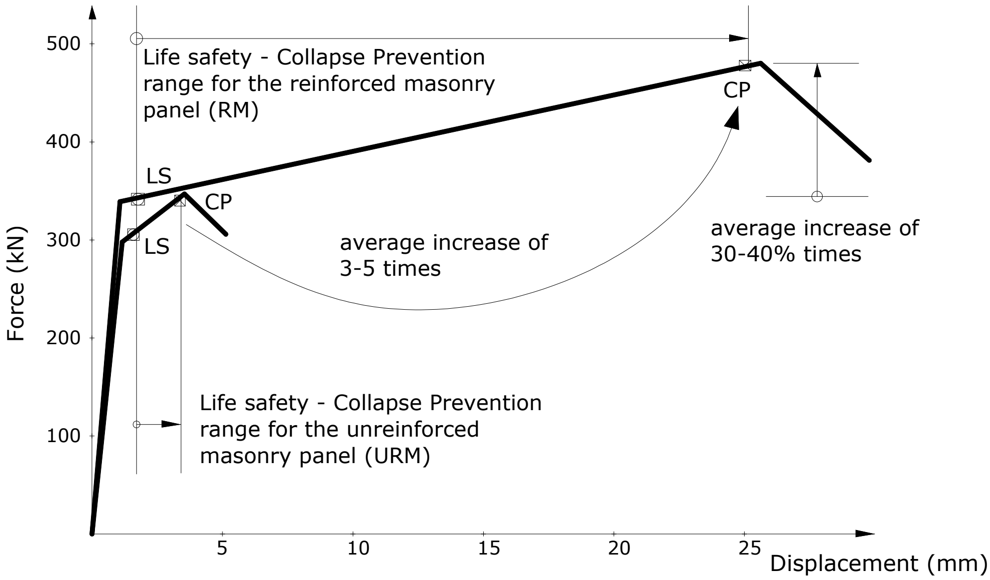


Figure 6-5. Benefit of the reinforcing from the point of view of performance levels

A parametrical study has been performed in order to establish the performance levels,. In this study, a "numerical experimental procedure" was applied (see Figure 6-6) aiming to observe the effect of the retrofitting solution in the case of an old masonry with the mechanical characteristics presented above, expressed in terms of the variable wall thickness ranging from 350 to 600 mm.

Tests on wall specimens have concluded that this technique improves the behaviour in the range of Life Safety – Collapse Prevention, along with an increase in strength.

These results allow the use of an equivalent material model for masonry, removing the tension softening (see Figure 6-7), so as to obtain the same global behaviour as in case of "numerically tested" retrofitted specimens. This observation simplifies a lot the numerical effort, by a simple change in the original material parameters in order to replicate the beneficial effect of reinforcing (see Figure 6-8).

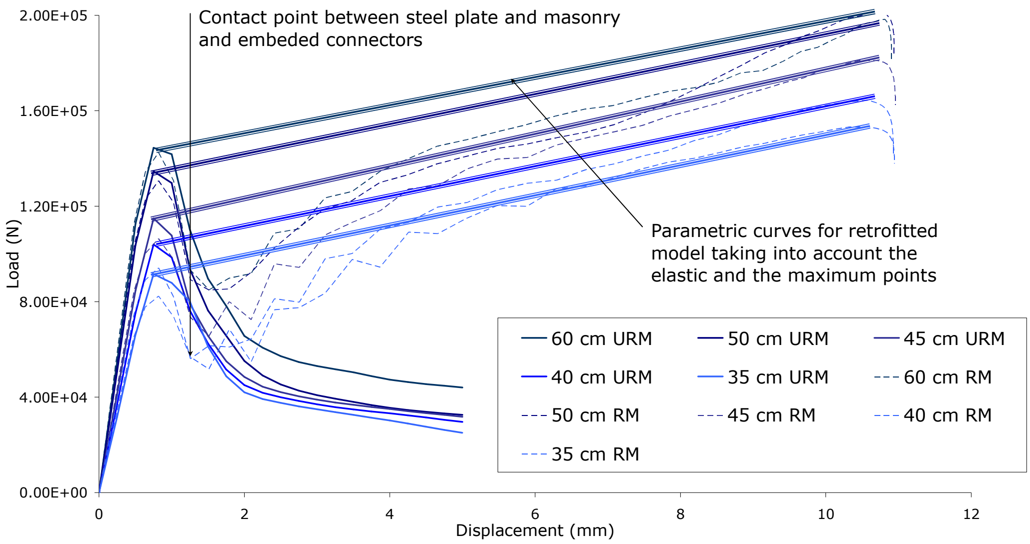


Figure 6-6. Numerical results for unreinforced and retrofitted masonry walls

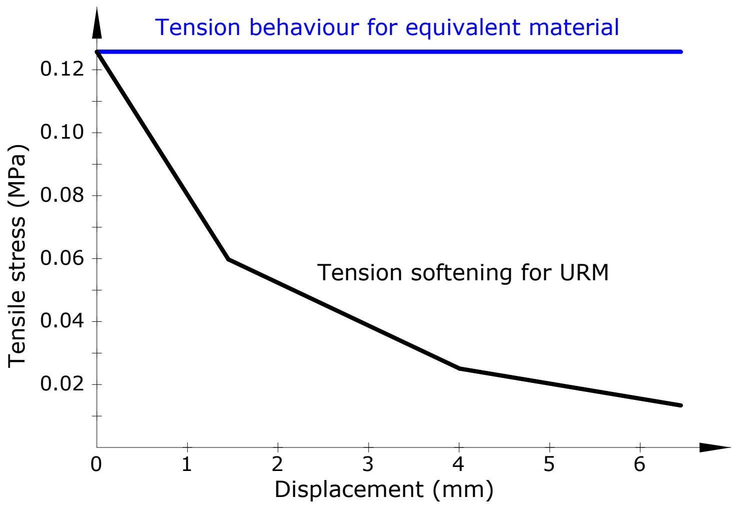


Figure 6-7. Material constitutive law in tension (tension softening – unreinforced model)

Such a procedure may be applied with success in the case of global analysis of real façades and for different building typologies, making the analysis easy and quick. The retrofitted model reached at CP level 2.5% ultimate maximum plastic strain and -0.7% ultimate minimum plastic strain. For the equivalent model, at the same displacement of 10 mm, corresponding to a 1/150 drift, there was recorded 1.5% ultimate maximum plastic strain and -0.07% ultimate minimum plastic strain. These values will be used in the further evaluation as reference criteria.

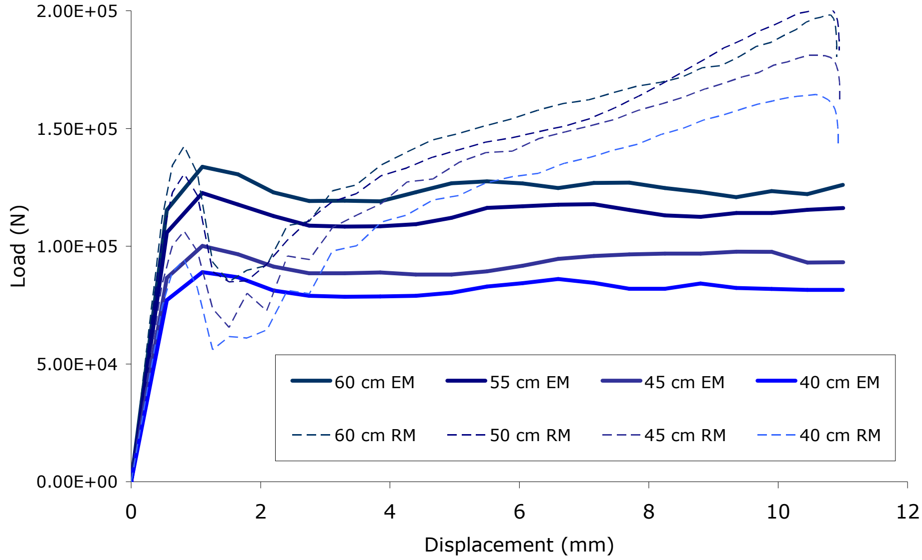


Figure 6-8. Comparative numerical results for calibrated models and equivalent material models

6.2.2. Numerical analysis of an existing building

Using ABAQUS code, a complete 3D model of the building has been built.

Some simplifications regarding the fixed base and rigid diaphragm behaviour of the floors have been used. The model is built from shell elements and a material model of Concrete Damage Plasticity was applied. The horizontal load was introduced quasi-statically, performing an explicit analysis, as force concentrated in the mass centre of the floors respecting a triangular shape, according to the first eigen vibration mode. The results of the pushover analysis are presented in terms of base shear force – top displacement (see Figure 6-9).

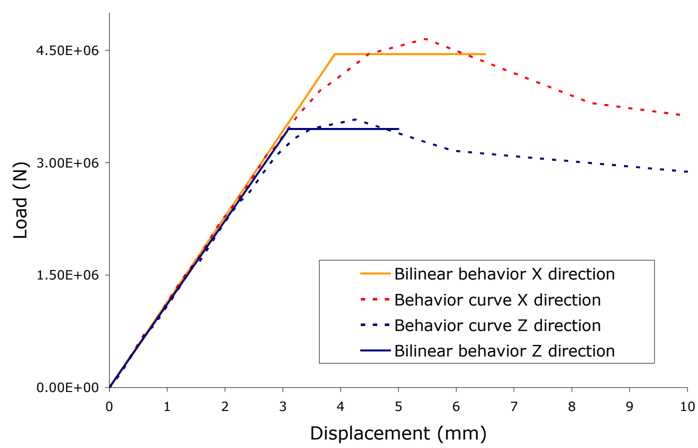


Figure 6-9. Global behaviour of the unreinforced building and the approximate elasto-plastic force – displacement relationship

Usually, after reaching the point of maximum force, then masonry building has a fragile and instable behaviour losing much of the strength at small displacement.

6.2.3. Performance analysis and evaluation

In order to establish the seismic response of both the initial and the retrofitted structure, a displacement based procedure was used [68],[66]. The target displacement has been determined at the intersection of capacity curve and the inelastic spectrum, considering a constant ductility of 1.5 (see Figure 6-10).

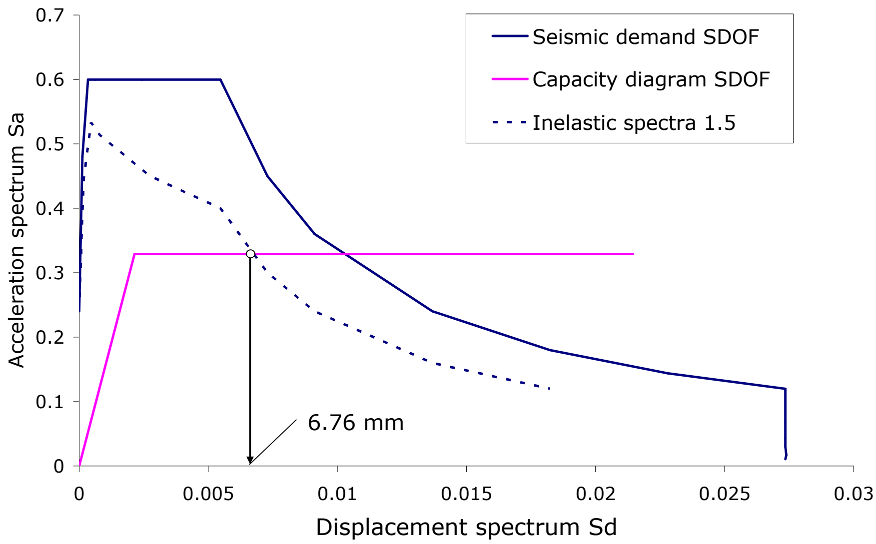


Figure 6-10. Demand spectra and capacity diagram for the unreinforced and reinforced numerical model

The damage level and the evidence of the attainment of the performance criteria at 9.77 mm target displacement for unreinforced model in terms of plastic strain is plotted in Figure 6-11.

We can see that for the un-reinforced model at the level of the ground floor, all the diagonal cracks were formed in between the openings and have exceeded the CP value of plastic strain (0.15%); consequently a soft storey collapse mechanism mode occurred.

At the attainment of 9.77 mm target displacement the retrofitted model, similar with the unreinforced building, the level of the reference plastic strain is exceeded in the unreinforced walls, but not in the reinforced ones (see Figure 6-14b). There can be concluded that, although failure occurred in the adjacent unreinforced walls, the reinforced walls are able to preserve the global safety of the building, by maintaining the same level of strength.

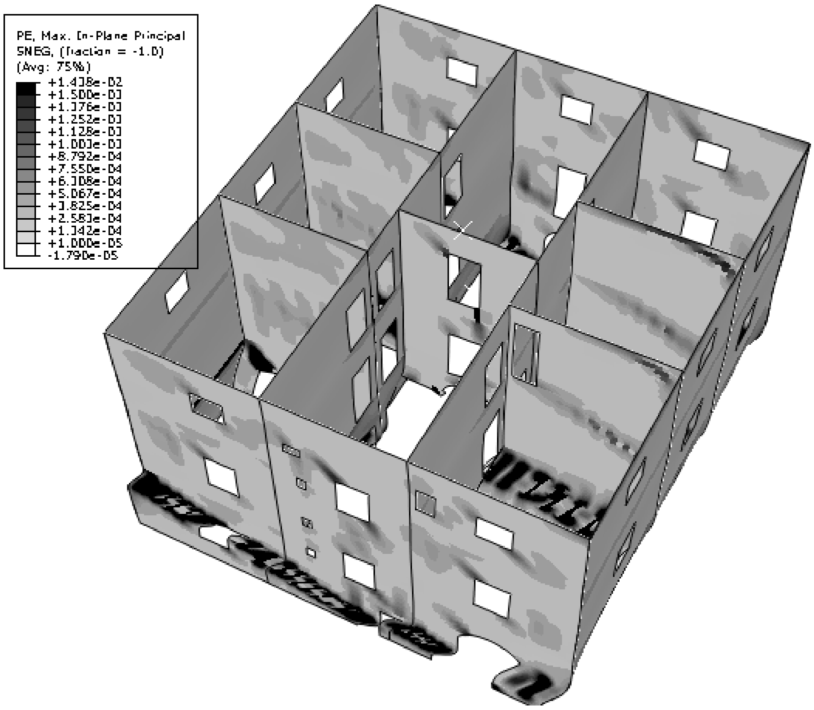


Figure 6-11. Areas where the plastic strain has exceed the collapse prevention level value

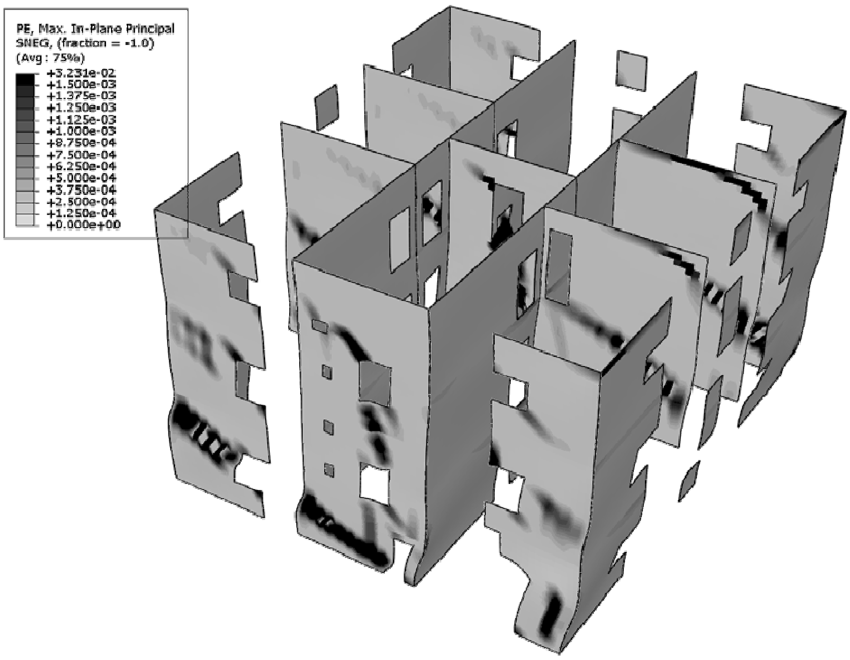


Figure 6-12. Retrofitted model behaviour plastic strain in unreinforced elements

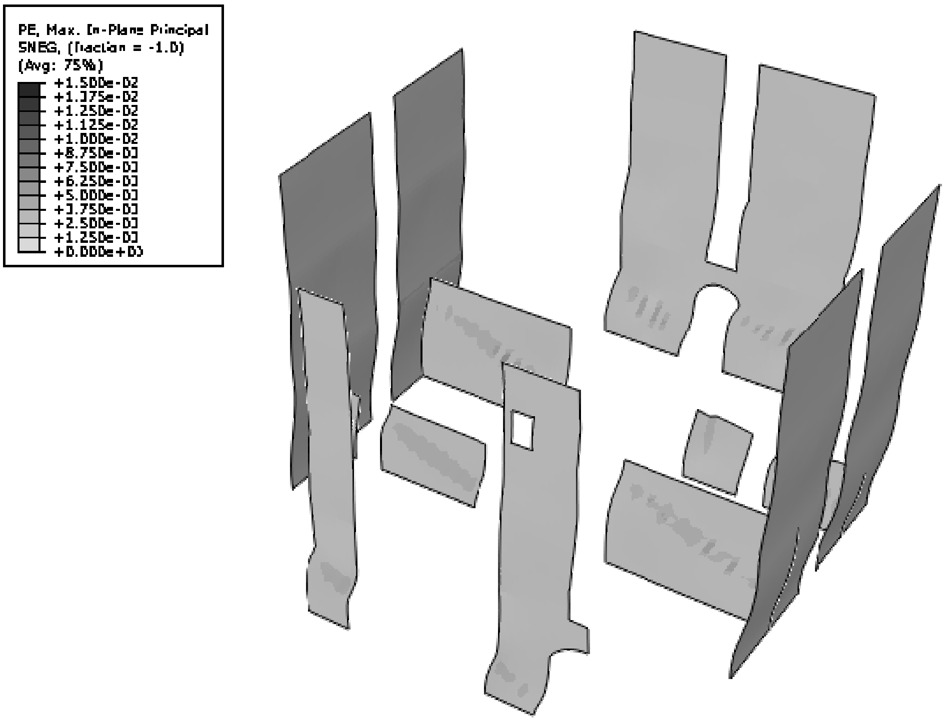


Figure 6-13. Retrofitted model behaviour plastic strain in retrofitted elements

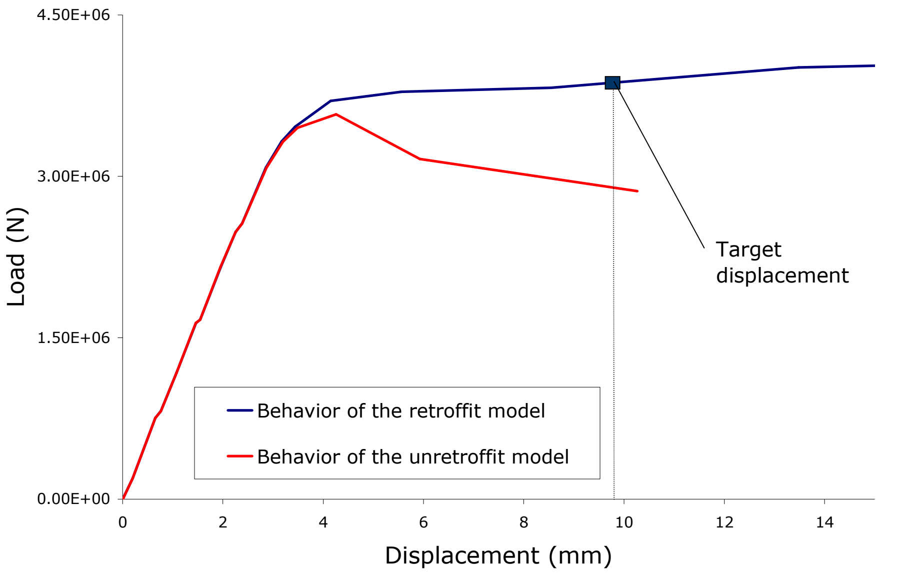


Figure 6-14. Comparative global behaviour of the unreinforced and retrofitted building

The retrofitted building subjected to a seismic motion of PGA up to 0.16g behaves in the elastic range and fulfils the IO performance level; for PGA between 0.16-0.44g, the LS performance level is attained. At a displacement larger than 30 mm, the building reaches the CP level. Using the recurrence formulas for PGA given in the Romanian Code P100-3 [171] even calibrated for Vrancea earthquake, a matrix may be built showing the performance objective possible to be achieved by a retrofitted building (see Table 6-1).

Table 6-1 Performance Objective

PL/IMR	30 y	50 y	100 y	225 y	475 y	975 y
PGA	0.072g	0.168g	0.24g	0.288g	0.36g	0.48g
IO	X					
LS		x	x	x	x	
CP						x

Such a matrix can be calibrated for other type of seismic motion, too.

However, in order to validate the equivalent material simplifications made in this case for global analysis, we need to extract the areas of important plastic strains concentration of the model and using the advanced numerical model to perform a new local analysis, respecting the geometry and boundary condition.

6.2.4. Concluding remarks

The present chapter proposes a "numerical experimentation" procedure for the analyses and evaluates the behaviour of the masonry structures retrofitted by metallic plates based on of performance criteria.

In the first step of the procedure, a stable and robust FE Model able to replicate the experimental observed failure mode and global behaviour was built.

In the second step, numerical simulations were performed for wall panels unreinforced and reinforced masonry, considering the real mechanical characteristics; the main advantages and benefits of the strengthening solution and acceptance criteria for the retrofitted elements have been obtained.

In the third step, equivalent materials that replicate the numerical results need to be determined. This approach allows for performing global analysis on real façades or entire buildings and assesses the damage at a certain seismic demand using a non-linear evaluation method, based on the acceptance criteria previously established.

In the fourth step, i.e. the validation, the most critical areas of the building must be selected in order to verify the local behaviour, by introducing relevant continuity conditions and using the calibrated model described at step one.

The retrofitting solution has showed a good behaviour, being able to preserve the initial capacity simultaneously by allowing for considerable ultimate displacement of approximately 0.7% drift ratio, which corresponds to the collapse prevention level of the building.

At the end, if applies the Decisional Matrix of Table 3-11 of Chapter 3 of this thesis, to scoring the intervention technique results is **8.33**. Thus according to this matrix, even better intervention techniques are possible, the one it was applied in present study is good enough.

Table 6-2 Decisional matrix

Structural aspects	L	M	H	Mark
Capability to achieve requested performance objective (after building evaluation!)			X	
Compatibility with the actual structural system (no need of complementary strengthening or confinement measures)		X		**9**
Adaptability to change of actions seismic typology (near field, far field, T<>Tic, etc)			X	
Adaptability to change of building typology			X	
Technical aspects	L	M	H	Mark
Reversibility of intervention		X		
Durability			X	
Operational			X	
Functionally and aesthetically compatible and complementary to the existing building		X		
Sustainability		X		**8**
Technical capability			X	
Technical support (Codification, Recommendations, Technical rules)		X		
Availability of material/device			X	
Quality control			X	
Economical aspects	L	M	H	Mark
Costs (Material/Fabrication, Transportation, Erection, Installation, Maintenance, Preparatory works)		X		**8**

Legend
L = low, M = medium, H = high
Mark – L (5-6), M (7-8), H (9-10)

7. CONCLUDING REMARKS. PERSONAL CONTRIBUTIONS

7.1. CONCLUSIONS AND PERSONAL CONNTRIBUTIONS

One of the most actual preoccupations in the civil engineering field is to preserve the historical heritage. Masonry buildings represent the overwhelming part of the European buildings, and beside their historical values many of them are still in use and fulfil different functionalities. Thus their preservation is important from the cultural point of view and they also have an important practical dimension. Because of the unsafe initial design, or of the long period of service, or because of earthquakes, many old buildings are nowadays in a poor state and need structural retrofitting. The applied retrofitting solution should have a minimal impact on the retrofitted building and has to be easily removed if a better solution becomes available and to meets the structural demands. This request is perfectly met by metallic materials. A short overview and a critical discussion about the actual practice in the field of structural retrofitting have emphasized the need of urgent intervention and underlined some important demands. Although the traditional technologies are still widely used, some new and innovative techniques based on modern materials are needed and have lately found their application.

This thesis proposes two retrofitting solutions of the masonry shear-walls based on the use of metallic materials. The proposed retrofitting techniques are new and innovative and attempt to fulfil two major concepts of the modern retrofitting philosophy, i.e. reversible solutions and the use of mixed technologies. The reversibility of the techniques is obvious, as they use "dry" connections for metal sheathing and steel wire mesh can also be easily removed by the heating of the resin.

A complete experimental program from material tests and calibration tests on small specimens (42 specimens of 500x500x250 mm) to real scale elements (22 specimens of 1500x1500x250 mm) was carried out in order to validate the techniques. An ingenuous testing set-up was build for tests in cyclic loading condition.

The experimental and numerical works have proved a good behaviour of the retrofitted elements especially in the range of Life Safety – Collapse Prevention, damage control domain, by allowing an important increase of the ultimate displacement. By improving the ultimate displacement, assuring a confining effect, like an external reinforcing able to carry tension some of the major disadvantage of the masonry are reduced. Not without importance is the fact that the system doesn't change much in terms of the initial stiffness, thus eliminating an important shortcoming of traditional retrofitting techniques. A very detailed numerical model has allowed to perform parametric analysis in order to assess the seismic safety of a historical building on the basis of a performance based approach. The numerical simulation in this study case has shown an important improvement of structural safety. A decisional matrix for retrofitting techniques validation that combines structural, technical and economical aspects, was applied in order to mark the proposed technique.

In order to practical apply these techniques, an evaluation methodology is proposed. A complete numerical tool calibrated based on experimental results is needed. By using the numerical model a “numerical experimentation” must be performed in order to establish the behaviour of different retrofitted walls typologies. An equivalent material able to replicate the numerical behaviour of the retrofitted model will be calibrated so as to be applied in global analysis of the buildings. A final validation is needed by extracting the critical areas of the building and making a local check, using the complete numerical model.

Taking into account the inhomogeneous nature of the masonry and the complexity of the composite retrofitting system, at which adds the scattering in the masonry behaviour, it is very difficult to obtain a robust numerical tool in oreder to relay on the results. Moreover, the analytical formulae are often unable to catch the real behaviour. In these cases, only experimental based design can be applied in order to predict and quantify the system behaviour. The thesis present an experimental based methodology, applied in the case of shear walls, for quantifying the effect of the retrofitted buildings.

The techniques have shown good behaviour and can be successfully applied in the case of masonry walls. The retrofitting solution based on masonry sheathing with metallic plates especially enhances the ultimate displacement and the steel wire mesh improves the resistance of the walls. By a proper calibration regarding the connector spacing, the steel grade and the plate thickness, the solution can offer a good behaviour of the retrofitted elements, and, by rational positioning, an optimal response of the entire building. The reported results are about pure shear behaviour of the panel, but take into account the combined failure mechanism the displacement of the walls is much more significant. By supplementary measurements in order to reinforce the toe area subjected to compression or by placing rubber elements, a more important improvement can be obtained. The connection of the metallic sheathing with the adjacent elements will significantly increase the positive effect of the proposed retrofitting techniques.

Main contributions of this thesis may be summarized as follows:

- Proposal of two innovative techniques for retrofitting masonry walls(new ideas);
- A complete experimental program from calibration test to validation on large specimens (a new testing set-up for the cyclic tests);
- The adaptation of a numerical model built referring to the all details of the retrofitting technique;
- The practical application in the study case.

7.2. DISSIMINATION OF THE MAIN RESULTS AND FUTURE DEVELOPMENTS

The Ph.D. thesis subject was developed around the PROHITECH European research grant and the entire experimental program and numerical investigations were made in relation to its specific targets and working groups. The performance based assessment of the case study was done within the framework of the RFCS grant STEELRETRO. The thesis was also supported by a doctoral grant (i.e. TD 407/2007 Study on seismic retrofitting techniques based on metallic materials of RC and/or masonry buildings - director) given by the CNCSIS/UEFISCU Romania Authority. Other research grants that have supported the thesis research, especially in conection with structural behaviour at seismic action, are:

- 32940/2004. Grant E, Tema 3, cod CNCSIS 31 Experimental stand for cyclic tests (Director Prof. Dan Dubina);

- 32940/2004 Grant A, Tema 10, cod CNCSIS 167 Metallic based techniques for consolidation and rehabilitation of buildings located in seismic areas (Director Prof. Daniel Grecea);
- CEEX-ET 60/2005 Numerical and experimental study of connection system of composite steel-concrete structures located in seismic areas (Director assoc. prof. Adrian Ciutina);
- CEEX-MATNADTECH – 29/2005 Study of strength and ductility performances of high-strength steel used for seismic structures (Director Prof. Dan Dubina);
- CEEX-ET 1436/28.03.2006 Dual steel structures, with removable dissipative elements, located in seismic areas (Director assoc. prof. Aurel Stratan);
- 2004-2005 Bilateral grant Romanian – Greek: Conservation and rehabilitation of monumental and historical buildings with technologies based on light gauge steel (Director Prof. Dan Dubina);
- 2006-2007 Bilateral grant Romanian – Greek: Strengthening and rehabilitation of historical buildings by reversible technologies (Director assoc. prof. Aurel Stratan);
- 04/15.09.2006 (2006-2008) Platforms and laboratories for interdisciplinary research and formation: Research Centre for Advanced Studies and Research on Material and Structural Engineering (CESCIMS) (Director prof. Dan Dubina);
- 3175/2007 – Research grant PNCDI II - frame 4: Structural systems and innovative structural technologies for extreme load protection of buildings in frame of durable development (Director prof. Daniel Grecea).

The author has presented the results of the research between 2005-2009 within the framework of the PROHITECH meeting from: General Meeting – June 2005, Ponta Delgada; WP7 Meeting - September 2005, Naples; General Meeting – November, 2005, Heraklion; WP8 Meeting – March, 2006, Naples; General Meeting – September, 2006, Poiana Brasov; General Meeting – April, 2007, Liege, General Meeting – January, 2008, Antalya and General Meeting – August, 2008, Sharm-El-Sheikh.

During several meetings held in Timisoara – October 2008 and Frankfurt – February 2009, in the frames of STEELRETRO, there was presented the performance based approach and the results on masonry benchmark.

The research activity has been applied in five articles publish in Romanian scientific journals, five articles in international conference volumes and four articles and datasheets in COST volumes and several conference presentations related to the Ph.D. topic.

Romanian journals:

- Dogariu A. – Models for calculation and analysis of masonry shear walls – Buletinul Stiintific al Universitatii "Politehnica" Timisoara, Romania – Seria Constructii si Arhitectura – Vol54(68), Fascicola 2, 2006; pag 95, ISSN 1224-6026;
- Dogariu A., Masonry walls strengthened with metal sheathing: FEM modelling, Buletinul Stiintific al UPT - seria Constructii-Arhitectura, Tomul 53 (67), Fascicola I, 2008, ISSN-1224-6026;
- Dubina D., Grecea D., Dinu F., Stratan A., Ciutina A., Dogariu A. – Performance based design of steel structures – Sesiunea stiintifica festiva prilejuita de implinirea a 60 ani de catre d-nul Prof. Dr. Ing. Radu Bancila" – ed. Solness Timisoara, 2005, pag. 137. ISBN 973-729-035-6;

- Grecea D., Bordea S., Stratan A., Dogariu A., Dubina D. - Solutii moderne pentru consolidarea si reabilitarea cladirilor amplasate in zone seismice - 2/2007 Buletinul AICPS, pag. 2 - 14, ISSN 1454-928X;
- Grecea D., Bordea S., Stratan A., Dogariu A., Dubina D. - Solutii moderne pentru consolidarea si reabilitarea cladirilor amplasate in zone seismice - Structuri metalice amplasate in zone seismice - Preocupari actuale - ed. Orizonturi Universitare, 2008, pag.141, ISBN 978-973-638-377-9.

International conferences:

- Dubina D., A. Dogariu, A. Stratan, V. Stoian, T. Nagy-Gyorgy, D. Dan, C. Daescu – Masonry wall strenghtening with inovative metal based techniques – International Conference on Steel and Composite Structures ICSCS07, Manchester, UK , Steel and composite structures, editors Y.C. Wang & C.K. Choi, pag. 1071, ISBN 978-0-415-45141-3;
- Nagy-Gyorgy T., Stoian V., Dan D., Daescu C., Demeter I., Diaconu D., Dogariu A. – Experimental assessment on shear strengthening of clay brick masonry walls using different techniques (accepted PROHITECH09 conference);
- Dogariu A., Dubina D & Campitiello F., DeMatteis G. – Experimental based calibration of a FE Model for numerical analysis of masonry shear panels strengthened by metal sheathing (accepted PROHITRECH09 conference);
- Dogariu A., Dubina D. - Performance Based Seismic Evaluation of a Non-Seismic Masonry Building of Metal Sheathed Walls. Part I: PBSE and Intervention Strategy (accepted PROHITRECH09 conference);
- Dogariu A., Dubina D. - Performance Based Seismic Evaluation of a Non-Seismic Masonry Building of Metal Sheathed Walls. Part II: Study case (accepted PROHITRECH09 conference).

COST publications:

- Dogariu A., A. Stratan, D. Dubina, T. Gyorgy-Nagy, C. Daescu, V. Stoian – Strengthening of masonry walls by innovative metal based techniques – COST 26 – Urban Habitat Construction Under Catrastrophic Events –Proceedings of Workshop in Prague, 30-31 Martie 2007, pag. 201-210. ISBN 978-80-01-03583-2;
- Bordea S., A. Stratan, A. Dogariu, D. Dubina – Seismic upgrade of non-seismic r.c. frames using steel dissipative braces - COST 26 – Urban Habitat Construction Under Catastrophic Events –Proceedings of Workshop in Prague, 30-31 Martie 2007, pag. 211-220. ISBN 978-80-01-03583-2;
- Dogariu A., D. Dubina - Performance of masonry shear walls strengthened with steel and aluminium sheeting - datasheet no. 2.20 - Urban Habitat Construction under Catastrophic Events - COST 26 - Editors: Mazzolani, Mistakidis, Borg, Byfield, De Matteis, Dubina, Indirili, Mandara, Muzeau, Wald & Wang - ISBN 978-99909-44-40-2 (paperback); ISBN 978-99909-44-42-6 (Hardback) - pg. 229-234;
- Dogariu A., D. Dubina & F. Campitiello, G. De Matteis - FEM Modeling Masonry Shear Walls strenghtened with metal sheating - datasheet no. 2-21 Urban Habitat Construction under Catastrophic Events - COST 26 - Editors: Mazzolani, Mistakidis, Borg, Byfield, De Matteis, Dubina, Indirili, Mandara, Muzeau, Wald & Wang - ISBN 978-99909-44-40-2 (paperback); ISBN 978-99909-44-42-6 (Hardback) - pg. 235-240.

The author have present to several international (7) and national (9) conferences, symposiums and seminars preliminary results of the research:

- Conservation and rehabilitation of historical buildings and monuments by using light gauge metal structural elements – 18 April 2005, Thesalonki, Grece;
- First international PhD Symposium, 20-21 September 2005, Pecs, Hungary;
- Strengthening and rehabilitation of historical buildings by reversible technologies – 22 February 2007, Thesaloniki, Grece;
- 3rd ICSCS 2007 Manchester, 30 July - 1 August 2007, Manchester, Great Britain;
- „Third International PhD Symposium in Engineering" - 24-27 Octomber 2007, Pecs, Hungary;
- Risk on Cultural Heritage (RICH) - Seminar - 31 august 2008, Cairo, Egypt;
- Symposium on "Urban Habitat Construction Under Catastrophic Events" Malta, 23-25 October 2008;
- Academics Days –22-23 May 2003, Timisoara, Romania;
- Quality Week AGIR – November 2003, Timisoara, Romania;
- AICPS Conference, 8 April 2005 ,Timisoara, Romania;
- PhD students Conference - STUDENT, 22 April 2005, Cluj-Napoca, Romania;
- National Conference AICPS, 19 May 2005, Bucharest, Romania;
- Academics Days – 26-27 May 2005, Timisoara, Romania;
- PhD students Conference, 11 May 2007, Cluj-Napoca, Romania;
- Academics Days, 25 May 2007, Timisoara, Romania;
- Quality Week, Modern techniques for seismic rehabilitation of historical buildings, 3-7 November 2008 Timisoara, Romania.

During the PhD stage, the author of this thesis have take part to summer academies, traings and short time scientific missions at: Bauhaus-Universitat Weimar, Germany – Europäische Sommerakademie ESA-2003 Advanced studies in structural engineering and CAE (2003); University "Federico II" Naples, Italy – Short Time Scientific Mision (STSM) – COST 12 – Improving of buildings structural quality by new technologies - European Science Fundation (prof. Federico Mazzolani) (2004); University "Federico II" Naples, Italy, ERASMUS – SOCRATES scholarship (2006); University "Federico II" Naples, Italy – Short Time Scientific Mision (STSM) – COST 26 – Urban Habitat Construction Under Catastrophic Events - European Science Fundation (2007); CNRRS-Bucharest - Training Program on Seismic Risk, organized by National Center for Seismic Risk Reduction (NCSRR) and Japan International Cooperation Agency (JICA).

The research on metallic based techniques for retrofitting masonry buildings will be continuated in the frame on FP7 European Grant MERLIN. Also future developments regarding the application of the proposed techniques in case of weak reinforced concrete diaphragms are also of interest. Off course a more appropriate for practical design, analytical approach can be developing.

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