NEW INTEGRATED KNOWLEDGE BASED
APPROACHES TO THE PROTECTION OF CULTURAL
HERITAGE FROM EARTHQUAKE-INDUCED RISK

NIKER
Grant Agreement n°
244123

Deliverable 3.3

Critical review of methodologies and tools for assessment of failure mechanisms and interventions

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WORKPACKAGE 3: Damage based selection of technologies

Leader: POLIMI

PROJECT N°:   244123

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TITLE: **New integrated knowledge based approaches to the protection of cultural heritage from earthquake-induced risk**
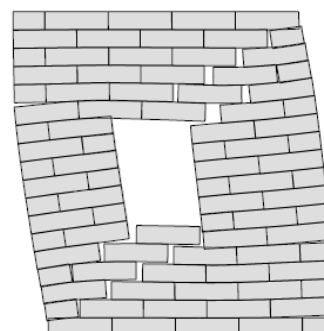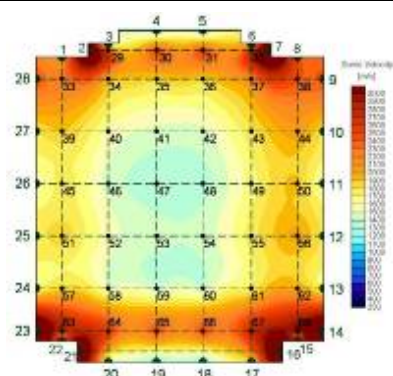
COORDINATOR: Università di Padova (Italy)

START DATE: 01 January 2010   DURATION: 36 months

INSTRUMENT: Collaborative Project
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THEME: Environment (including Climate Change)

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ANNEX 1 - CASES STUDIES

ANNEX 2 - TEMPLATES OF DAMAGE

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1 INTRODUCTION

Historic buildings, no matter whether they are famous monuments or so called "minor" or even vernacular, architecture represent an important part of our cultural heritage. This patrimony which is the living memory of the country history and development must be preserved as much as possible as an historic document of our past. Unfortunately wars and dramatic events (earthquakes, floods, slides, fires, etc...), but also abandonment and lack of maintenance are constant menaces to the cultural heritage in every country of the world.

Many experiences in restoration and rehabilitation of damaged masonry buildings have been accumulated in Europe as in other countries during more than fifty years. In some cases reconstructions of destroyed monuments and historic centres were the consequence of wars (e.g. the reconstruction of the centre of Warsaw after the Second World War). In others, restorations were carried out after a long period of misuse and lack of maintenance following the same war.

During these decades also major earthquakes have taken place in most of the Mediterranean countries, destroying monuments and dwellings and causing the demolition of badly damaged constructions or the quick reconstructions of dwellings and villages which were wrongly considered as minor patrimony, on which invasive intervention could be made.

In the last decade the term "restoration" has more and more been substituted by the term "conservation", meaning that the historic buildings should first of all be preserved as much as possible; this preservation need has also been extended to the minor architecture, considering the historic centre as a monument on its own.

Also in the case of damages due to earthquake or other calamities the expression "to adequate" meaning the possibility of invasive interventions in order to reach the safety coefficients adopted for new buildings was substituted by the expression "to improve by minor repair and strengthening", (Corsanego 1992), (Corsanego 1993) and (Gavarini 1994). This policy was assumed when failures of some repair and strengthening techniques were seen after further earthquakes. In fact, historic buildings which were adequate to the modern safety coefficient adopting models used for new buildings showed partial or total collapse during recent earthquakes due to the incompatibility of interventions based on modern materials and techniques (Borri 1999a), (D'Ayala 1999b), (Binda 1999a,b), (Binda 2003c) and (Binda 2005a).

In particular, the 1997 seismic sequence in Italy gave birth to a deep re-thinking of the theoretical basis of seismic vulnerability of historic masonry buildings, which led to the introduction of a new seismic code.

The fact that several buildings were repaired and/or strengthened prior to the earthquakes, allowed the assessment of the applied intervention techniques. In addition to the typical damages observed in numerous historic structures, the effects of the seismic events showed that, in several cases, the adopted structural models, (that presumably described the structural system of the historic construction), were not adequate and the retrofitting techniques had not provided the expected effects. Actually, the earthquakes have revealed the cases of incompatibility between the existing structure and the way the intervention technique was applied. This incompatibility is attributed to insufficient knowledge of either the bearing system or the properties of the on-site materials. The lack of sufficient documentation has led to the selection of inadequate repair/strengthening techniques, (Figure 1.1).

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Figure 1.1 Example of a) Out-of-plane collapse of a wall with R.C. tie beams, b) roof hammering the masonry walls, (Binda 2006a).

Most of the failures were due to lack of knowledge of the materials and building construction details which caused a wrong choice of the repair technique. Furthermore, in several cases, poor application of the selected techniques was observed due to the lack of knowledge and of skill. Although these observations call for regulatory documents that would provide the Engineer with adequate guidance, the complexity of the subject, (a multi-parameter problem with social, historic, aesthetic, technical and economic aspects), does not allow for the definition of general rules and operative modalities, as it was tempted in the past.

The observation of failures of repaired buildings due to incompatibility between the original structure and the repair, showed the necessity of developing new structural models for the old masonry buildings and code requirements for the intervention, (Borri 1999a), (D'Ayala 1999a), (Magenes 2000) and (Modena 2004a). The code requirements were oriented previously to a concept of seismic adequacy of the structures, (LLPP 1996).

The new Italian seismic code moved from theories of "adequacy" to "improvement", (PCM 2003), (PCM 2005) and (LLPP 2008, 2009), which meant more compatible and respectful interventions on the historic building patrimony, (Corsanego 1992), (Corsanego 1993) and (Gavarini 1994).

Following the previous issue, extensive damage surveys were carried out on damaged centres, in order to define the real structural behavior of the historic buildings. Based on this criterion new methodologies were proposed as an approach to the seismic behavior of these buildings.

Several studies based on on-site observations after seismic events allowed to arrange abacuses of the typical damages occurring to different typologies, (ex: buildings, churches, palaces), which led to the consequent systematization of mechanical models able to describe their specific behavior through kinematics models, both for in-plane and out-of-plane mechanisms. The Civil Protection Department and the Ministry of Cultural Properties in Italy have published damage survey templates with detailed drawings illustrating the most important crack patterns after the earthquake in churches, (Figure 1.2), and palaces, (PCM 2001). Other templates, with more simplified schemes were also prepared, (GNDT 1999), (Aedes 2000), (PCM 2000), for the post seismic damage survey and used in post-earthquake emergency time for the prompt safety evaluation of dwellings. The occurrence of a peculiar mechanism depends on the level and type of connection of the façade to the side walls.

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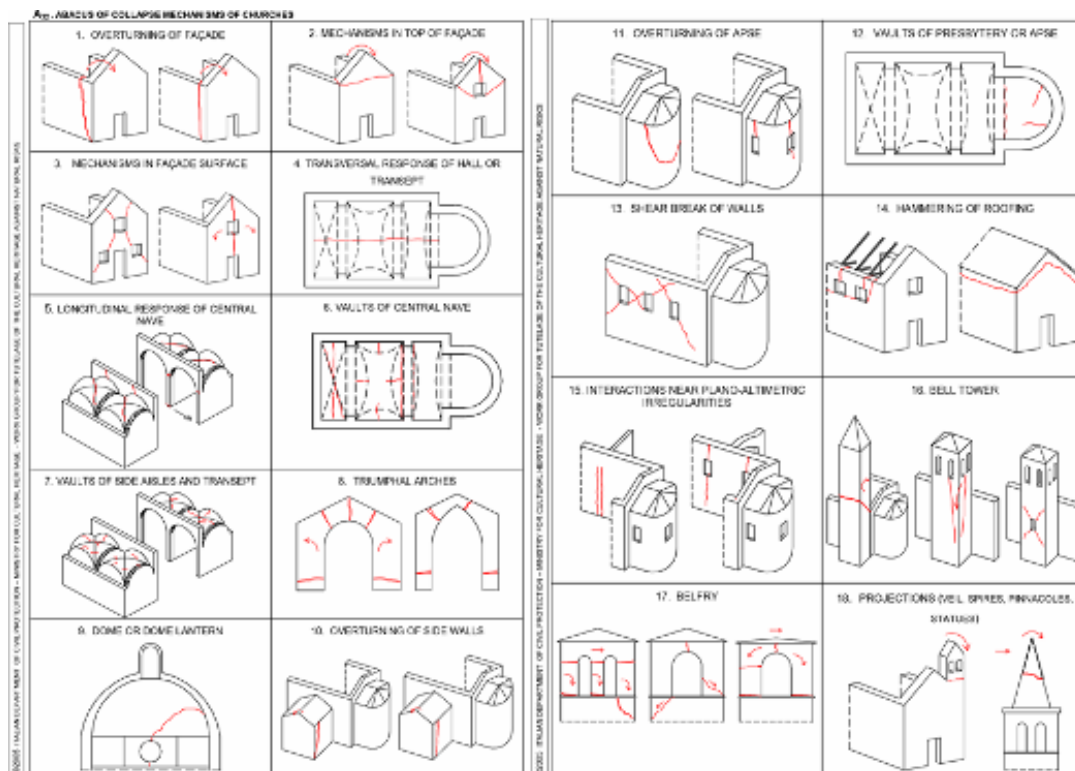
Figure 1.2 Survey templates with drawings illustrating the most important crack pattern after earthquakes in churches, (PCM 2001).

After the collection and interpretation of the damages that occurred on repaired buildings it can be clearly stated that most of the incompatibilities were due to the lack of knowledge of the construction techniques, of the material used and of the evolution of the structure along the centuries, (Penazzi 2000, 2001). Therefore it is possible to conclude that the knowledge of the construction should be really deep in order to understand the role of all its features and details, the characteristics of the materials and structure and its eventual evolution in time, (Modena 2004).

Prevention and rehabilitation can be successfully accomplished only if a diagnosis of the state of damage of the building has been formulated. Besides the preliminary damage controls and the investigation before the design for intervention, the effectiveness of the repair techniques should be controlled during and after the repair work, as well. The investigation also may require long-term monitoring of the structure, (Binda 2000).

The diagnosis should result (i) from an experimental investigation on-site and in the laboratory aimed to define the characteristics of the materials and of the structure itself and (ii) from the structural analysis based on appropriate mathematical models, (Binda 2007d). The investigation on-site must be non-destructive as far as possible and give information with good precision.

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1.1   DESCRIPTION AND OBJECTIVES OF THE WORKPACKAGE

WP3, in general, is aimed to the collection of information for increasing the existing state of knowledge linking earthquake induced failure mechanisms, construction types and materials, interventions, assessment techniques. According to the project document, other aims of the WP are the follow:

Development of concepts for materials and intervention techniques based on the structured database; definition of the main design parameters and requirements for materials and intervention techniques;

Definition of the main on-site control techniques and strategies;

Development of advanced materials and improved techniques for intervention and to produce/assemble those required for testing and case studies;

Development of laboratory procedures and choice of parameters for the final validation of the durability, compatibility, and effectiveness of new techniques and materials;

Parameterization of all the above mentioned information to set the basis of optimized design and required laboratory testing in following WPs.

1.2   OBJECTIVES AND STRUCTURE OF THE DELIVERABLE

The aim of the Deliverable D3.3 is to produce an inventory of tool for assessment of failure mechanisms and interventions.

Information was collected by the partner expertise and in literature in order to amplify the case histories.

The analysis of the information was carried out the base of the state of the art and methodology of survey, in order to recognise characteristic behaviors and recursive damages.

The deliverable starts from a general description of the methodology of survey and investigation, stressing differences between on-site and numerical assessment and real behavior surveyed after the recent earthquakes.

The analysis explores open topics within the international debate like the problem of the non destructive technique and the real features of the masonry structure. Concerning this topic are considered the most recent applications of new techniques.

Annex 1 include case studies on some different methodology of survey and investigation.

Annex 2 includes damage assessment template for cultural heritage and template for typological survey, masonry quality.

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2 METHODOLOGY OF SURVEY AND INVESTIGATION

Historic masonry buildings, whatever use is made of them at present or in the future, have to show structural stability. From the point of view of the risk for human life, they may belong to several categories from: (i) isolated buildings, (ii) buildings belonging to the urban area, (iii) buildings open to the public, and (iv) buildings open to large assembles of people, (cathedrals, theatres, etc...). For each of the mentioned categories a certain amount of risk, as it is for new buildings, has to be accepted, (Macchi 1992). An appropriate and rational use of the structural analysis can help in defining the eventual state of danger and in forecasting the future behavior of the structure. To this aim, the definition of the mechanical properties of the materials, the implementation of constitutive laws for decayed materials and of methods of analysis for damaged structures and the improvement of reliability criteria is necessary.

In the case of historic buildings the constitutive laws coming from a good knowledge of the material is not enough. In fact the classes of buildings above mentioned usually correspond to different typologies and to different behaviors: (i) isolated buildings, (ii) building in a row, (iii) complex buildings, (iv) towers, (v) palaces, (vi) churches, (vii) arenas, (Figure 2.1), (Binda 2003a). The modelling of these structures can be very difficult. In fact, when the structure is a complex one, only linear elastic models are easily usable. Non-linear models or limit state design complex models are difficult to apply, also because material constitutive laws are seldom available. Furthermore when the complexity of the structure is given by its evolution along the centuries starting from a simple volume to a more and more complex volume, (Figure 2.2), then modelling has to take into account all the vulnerabilities accumulated during the subsequent transformations, (Binda 2003a).

The same difficulties can be found in choosing the techniques for repair and strengthening. No doubt that the mistakes shown in Section 2 could have been avoided if a better knowledge of the materials and of the structure from its geometry to its modifications would have been known.

Figure 2.3 shows which information can be available from on-site and laboratory survey and how they can constitute the input data for the structural analysis. When the design of the survey is previously available, the conclusions from the experimental and numerical investigation will conduct to the diagnosis of the real state of the structure. The designer should remember that every investigation has its cost; it is evident therefore that every single operation must be carefully designed and optimised to obtain the desired results.

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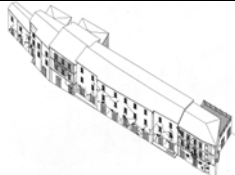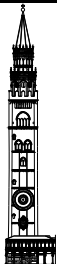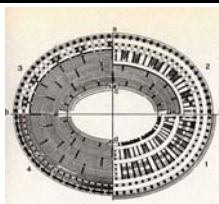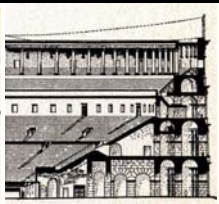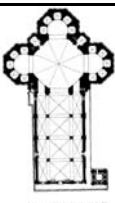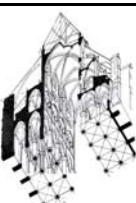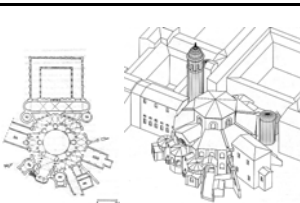
Type A) Isolated houses and/or dwellings	Type B) Row Houses	Type C) Palaces
Ms7 axonometric	Sel UMI 23 – axonometric view	Consoli Palace in Gubbio

Type D) Bell-Towers	Type E) Arenas	
The Torrazzo of Cremona	plan section	
	The Colosseum in Rome	

Type F) Churches and Cathedrals		
F1) Churches: plan based on latin cross scheme		**F2) Churches: central plan**
S. Maria del Fiore: plan	Gothic Cathedral	San Vitale: plan and axonometry

Figure 2.1 Building typologies, (Binda 2003a).
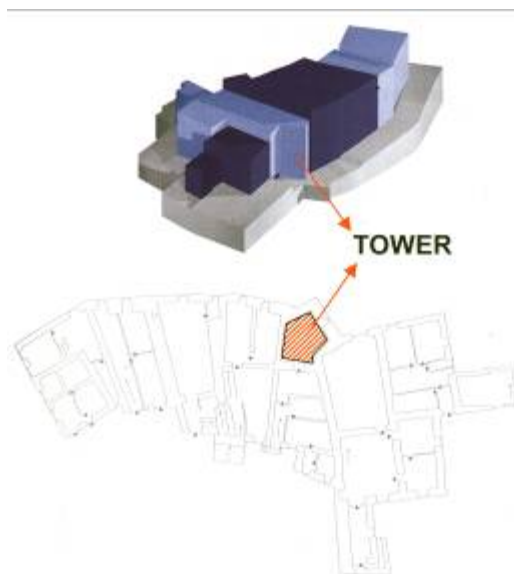
Figure 2.2 A complex building in a Umbria village and its evolution, (Cardani 2004).

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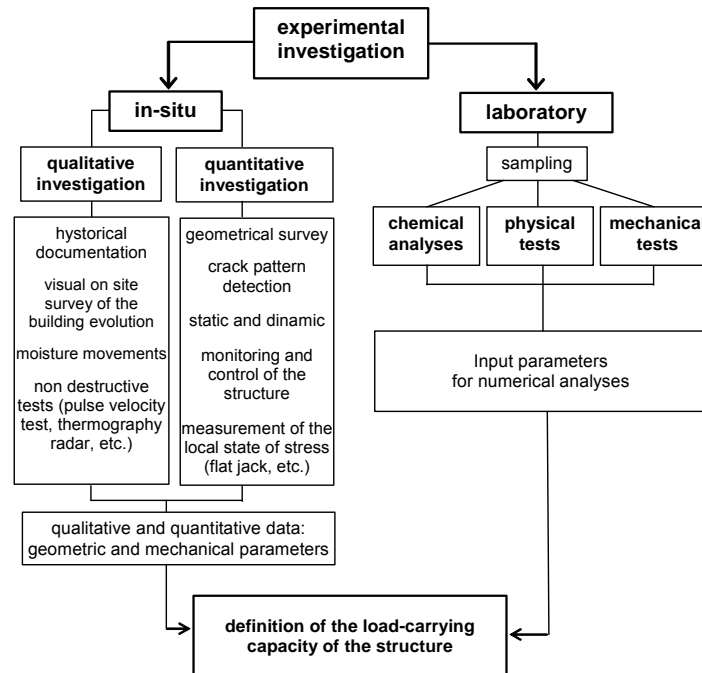
Figure 2.3 Finalisation of the experimental survey to the structural analysis, (Binda 1994).

The necessity of establishing the building integrity or the load carrying capacity of a masonry building arises for several reasons including: (i) assessment of the safety coefficient of the structure, (before or after an earthquake, or following accidental events like hurricanes, fire, etc...), (ii) change of use or extension of the building, (iii) assessment of the effectiveness of repair techniques applied to structures or materials, and (iv) long-term monitoring of material and structural performance.

The flow chart of Figure 2.4, (Binda 2000), schematically represents the needs to be fulfilled by the experimental investigation together with the techniques adequate to these needs. Non Destructive Techniques (NDT) can be helpful in finding hidden characteristics, (internal voids and flaws and characteristics of the wall section), which cannot be known otherwise than through destructive tests. Sampling of masonry specimens is a costly operation, which also can lead to misunderstanding when the operation is not carried out in the appropriate way.

The on-site mechanical tests available are flat-jack, hardness, penetration and pull out tests. The flat-jack tests give local measurements and are slightly destructive: nevertheless they can give directly the values of mechanical parameters.

Most of the ND procedure can give only qualitative results; therefore the designer is asked to interpret the results and use them as comparative values between different parts of the same masonry structure. It is also useful to use different ND techniques and compare the results between with the different techniques.

It must be clear that even if there is a need of consulting experts in the field, it is the designer, or a member of the design team, who must be responsible for the diagnosis and must: (i) set up the on-site and laboratory survey project, (ii) constantly follow the survey, (iii) understand and verify the results, (iv) make technically acceptable use of the results including their use as input data for structural analyses, (v) choose appropriate models for the structural analysis, (vi) arrive at a diagnosis at the end of the study.

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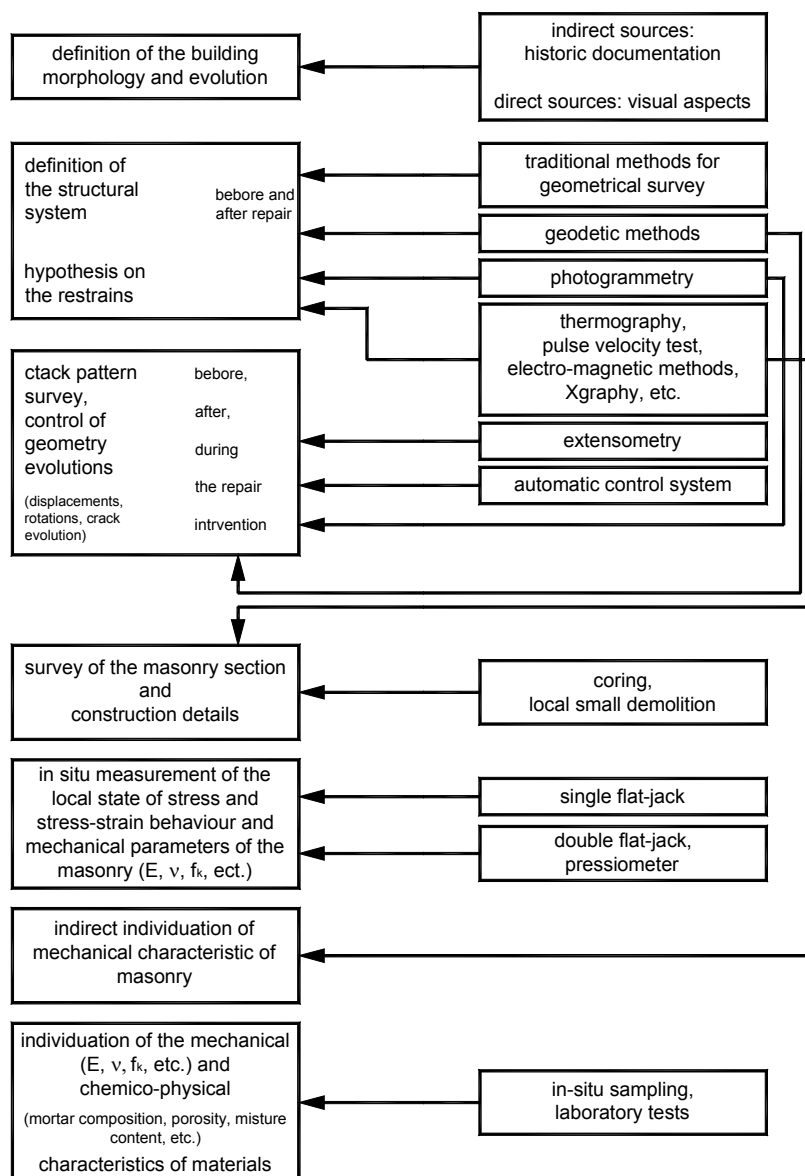
Figure 2.4 Information required and correspondent investigation techniques, (Binda 2000).

In the following section all of the most applicable procedures are briefly described taking into account their limits and advantages and also the incidence in the cost of the operations.

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3 GEOMETRICAL SURVEY AND HISTORICAL EVOLUTION

A diagnosis process usually starts with the reconstruction of the historic evolution of a building, accompanied by the direct survey of the building itself. The documentary research includes the collection of:

- Texts and accounts which describe the architecture of the building, its layout and plan, its use, the number of floors (storeys), the constitution materials used, a description of its environment, etc...
- Any old graphic documents, (plans, cross sections, additional works, land register, etc...), available in a municipality or registration offices.
- Old photographs, of the inside or outside of the building, which help checking the state of the construction at that time.
- Drawings, (sketches, watercolors, pencil drawings, etc...), old aerial photographs of a city, village or area in which the building could appear.
- Past Standards.

This research can lead to the identification of the original construction, its transformation and evolution: factors which define its architectural environment nowadays. Indeed, a building in its current state is the result of a continual change of uses, of a residential environment bearing the marks of the various inevitable interventions and changes brought by those living in the area.

3.1 METHODOLOGY FOR GEOMETRICAL SURVEY IN SEISMIC AREA

3.1.1 Identification of the building or aggregate (complex building) within the urban context

The identification and location of the building or of the aggregate within the urban context is the first action of the whole procedure concerning (survey).

Definition of complex buildings or aggregates

While a single building, (house, palace, church, etc...) is geometrically defined by its perimeter walls which clearly separate it from the other buildings, a complex building (or aggregate) is defined with more difficulty. It is possible to define an aggregate as a group of buildings, (each of which can be called single structural unit, Figure 3.1), connected one to the other through end walls to form an apparent continuous row. These connections can be structurally efficient or not depending on the historical evolution of the aggregate, nevertheless, the units interact during an earthquake or general dynamic actions.
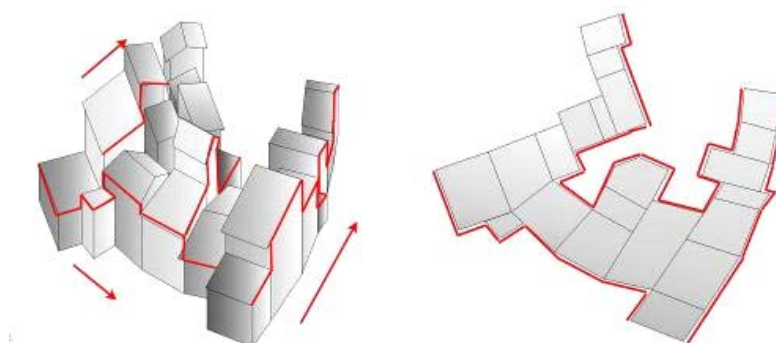
Figure 3.1 Changes of the stiffness due to the in plan and height irregularities of parts of the aggregate buildings, (Alberghini 2007).

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The evolution of the historic centres was characterised by a progressive occupation of the spaces between pre existing buildings, in adjacency with them or as a continuity, (Binda 2004a,b), (Binda 2005b,c), (Cardani 2004) and (Alberghini 2007). Nevertheless within an aggregate it is frequently possible to recognise the original and homogeneous buildings from which the evolution process of new volumes lead to the complete saturation of the existing free spaces. Sometime the aggregate can be constituted by a street block. In several cases the aggregates can be connected by arches and vaults which can influence their structural behavior. Their eventual contribution can be assessed by analytical models by inserting in their position concentrated actions or constraints in order to take into account local effects which do not influence the global behavior of the aggregate, (Valluzzi 2001), (Valluzzi 2005a,b).

The definition of the aggregate is followed by the individuation of the internal "homogeneous structural units" and by the elements which can create interactions between them. This operation is usually coincident with the historic evolution of the aggregate and can be valid also when detecting the damage after a seismic event.

In order to univocally detect the building or the aggregate it is important to insert it within the urban context map eventually using a GPS. When available, also the historic maps indicating the evolution of the historic centre, of the building and of the aggregate and the eventual collapses due to past earthquakes have to be added, (Figure 3.2, Figure 3.3, Figure 3.4 and Figure 3.5).

Figure 3.2 Campi Alto: supposed situation after the earthquake of 1703, (Cardani 2004).

Figure 3.3 Campi Alto: supposed situation after the earthquake of 1730 - Plan of 1820 (Brogliardo), (Cardani 2004).

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Figure 3.4 Campi Alto: supposed situation after the earthquake of 1859, (Cardani 2004).
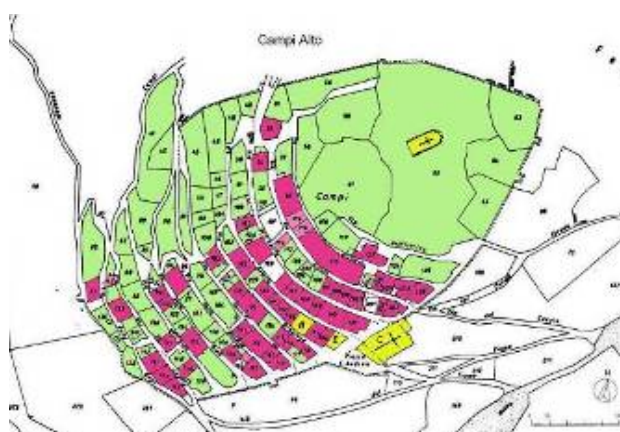
Figure 3.5 Campi Alto: situation in 2002, (Cardani 2004).

Hypotheses on the origin of the building or aggregate and historical evolution

The analysis of the evolution of the building or aggregate along the time is a very important step in order to check the vulnerable elements which can cause structural faults or effective structural connections.

The reconstruction of the historical evolution of the structure and of the subsequent volumes is essential to find the resisting structural system as a result of the continuous modifications.

This phase is useful to reconstruct or obtain through various hypotheses the chronological succession of the constructive phases of the building or aggregate. The aim is to find the original elements and the subsequent elements built in different times and their connections. In the case of aggregates this evolution represented on plans, prospects and sections is useful to find possible discontinuities in the walls, floors and roofs and also the variability in the morphology of the masonry walls, (variability in texture or in the section: one leaf, two or more leaves).

Hypotheses or definition of the evolution of the structure can be obtained through historical documents but also from accurate observation on-site, based on visual inspection.

Besides the historical reconstruction, this research is also essential to find vulnerabilities such as lack of connection between portions of the building or of the aggregate, (Figure 3.6). Also the definition of single properties or units can be of great help in this research.

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Figure 3.6 Lack of connection between portions of the building or of the aggregate, (Giuffrè 1993). A - Old cell of the aggregate; B - Following addition of a portion of aggregate: later addition; C - Following addition of a portion of aggregate: interclosed cell.

3.1.2 Visual inspection

A preliminary visual inspection is essential to understand the building typology, its position, if it is an independent building or an aggregate, its shape and dimensions, the macro elements and to design the geometrical survey deciding also the tools to be used for that.

The visual inspection allows also, together with preliminary advanced investigations, arriving to a good knowledge and to preliminary judgments on the quality of the structural elements, of the materials and their decay and, in general, on the causes which can influence the structural behavior.

The traditional structural simplified analyses are based on the direct visual inspection allowed by small local sampling, (elimination of renderings and plasters for small portions, removal of stones and bricks and mortars in order to investigate the section morphology, etc...). These small inside surveys allow examining the characteristics of the masonry on the surface and inside the section to see the connections between walls the constraints between floors and walls and between roofs and walls. When necessary also more refined non destructive techniques can be applied (ex: thermography, sonic tests, geo-radar, etc...) in order to observe on larger portions the homogeneity or non homogeneity of the walls.

The choice of the number, typology and position of these investigations is very important; this is why it was previously mentioned the concept "design for investigation", which has to take into account the variability of the building and features characteristics, but also the economical costs of every operation and the budget provided for the tests. In case of an aggregate it is important to recognise before visually the different volumes, their time of construction in order to sample out the position of the most representative area of investigation.

Of course the most important information to be collected is the typology and morphology of the load-bearing masonry walls. So when the masonry surface is completely hidden by renderings and plasters, it can be necessary to choose strategic points where to remove the rendering or plaster at least for 1.0m^2 avoiding of course positions with important renders, paintings or frescoes. These same points can be later on used for sonic and flat-jack tests (see below). The same operation of small removal has to be made in order to check the connection wall to wall, floor to wall, roof to wall.

Inspections have to be made also in order to detect the prospects of the walls, to find the distribution of windows and doors and the presence of the load bearing portions of the walls, (Figure 3.7). The same inspection should be made on the internal prospects.

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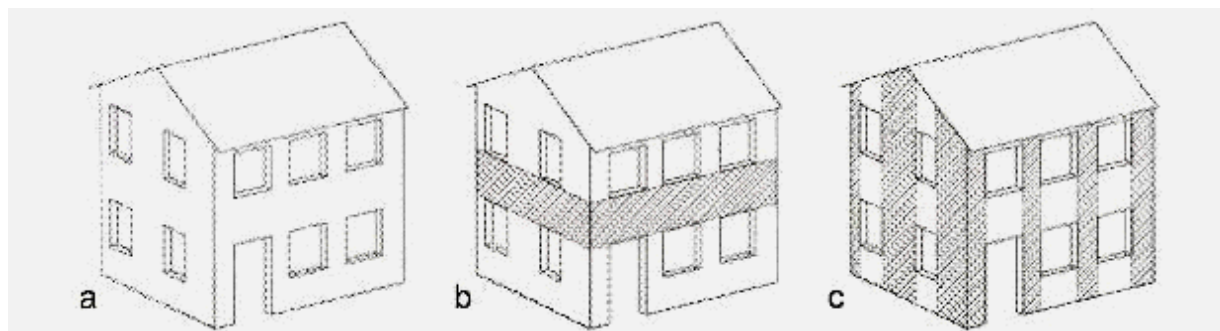
Figure 3.7 Diagram of the spandrels (b) and piers (c) in a building analysis, (Cardani 2004).

A Guideline for visual inspection especially dealing with maintenance, but which can also be used in general for damaged buildings, is being proposed and discussed within the RILEM TC 216 SAM, (TC 216SAM 2010). In some countries as in The Netherlands or in Belgium private organisation called Monumentenwacht are active and in connection with public and private owners periodically visiting the C.H. buildings before and since soon after the conservation intervention. They make a detailed report on the state of conservation of the building and carry out the small maintenance interventions or call for experts if a serious damage is surveyed.

Figure 3.8 International cooperation: visual inspection of the Abbey of Fontevraud in Saumur (France) by the Dutch Monumentenwacht.

3.1.3 Geometrical survey

The first step to the knowledge of the building and aggregate structural system is represented by the phase of the geometrical survey. The geometrical survey must report all the elements necessary to represent the development in plan and height of the building or aggregate. In the case of aggregates this phase is particularly important to highlight the spatial-topographical articulation of the adjacent connected buildings or structural units, (Figure 3.1). The geometrical survey should locate the exact position of the load-bearing walls at each floor also referred to the lower floor and the value of the thickness of each wall.

The optimal scale is 1:100 for the preliminary presentation, but also 1:50 for better details. This way it is possible to represent the geometry of all the elements defining the spaces, the load-bearing and non-bearing elements, the profile and dimension of the floors, vaults, roofs, staircases and the presence of specific elements, (niches, flues, cavities, etc...).

The geometrical survey will be the starting base on which the structural model will be developed, therefore, also all the other important information such as the position and height of doors and windows, the level of the floors, the spring of arches and vaults and domes in order to calculate the

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volume and the weight of all the elements, has to be given. Also deviations from verticality and horizontality have to be surveyed.

The survey will be given through plans, prospects, sections and eventually also 3D representations with details and pictures (also for aggregates).

3.1.3.1 Topographic survey

The geometrical survey can be carried out in the simplest and cheapest way by using the traditional topographic methods, with manual instruments and/or by a total station. A first base of fixed points constituting a closed polygonal perimeter can be stated to which all the measurements have to be referred. It is also important to measure by the total station all the deviations from the verticality and horizontality. An example of this determination is reported below with reference to the St. Lorenzo church in Cremona, (Anzani 2007).
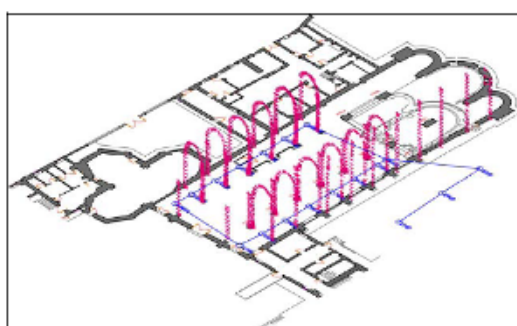
Figure 3.9 Topographic profiles, (Anzani 2007).

To determine the verticality of the pillars of St. Lorenzo Basilica at Cremona (Italy) and of the longitudinal walls at different sections a topographic survey was conducted using a laser integrated theodolite, (GEOTOP). Initially, a topographic network (local system) was created, formed by a closed polygonal made of 15 vertexes and constituting the framework of the survey, (Anzani 2007). The detailed survey of each pillar was based on the topographic network, thereby, minimizing the inherent error in measurement, (Figure 3.9).

An overall southward tilting characterizes all the pillars, with the exception of pillar C6b, (Figure 3.10a). This pillar tilts 15.0mm northward and is not connected by a tie rod to the south wall. The maximum displacement of the southward tilting pillars is 78.0mm (C3a) and 62.0mm (C2a). The displacements of the tie rods due to the pillar tilting is visible in Figure 3.10b, together with the values of the tie rod tension.

(a)

(b)
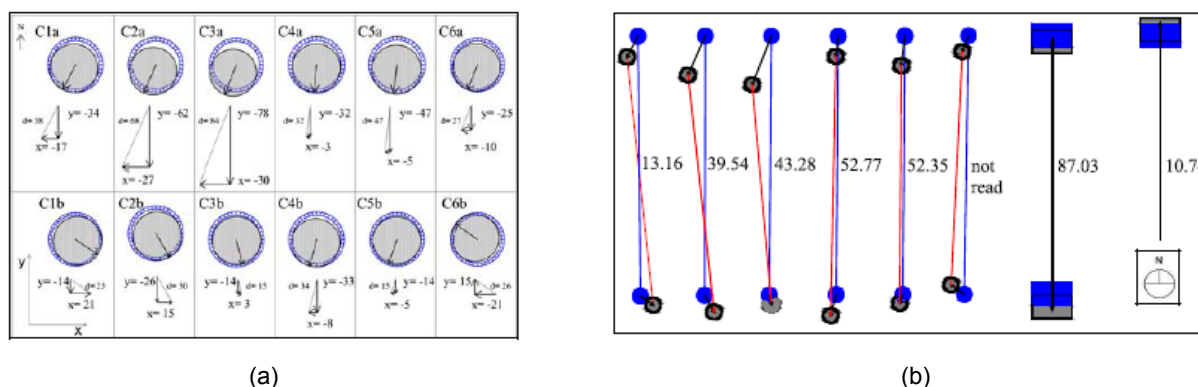
Figure 3.10 (a) Horizontal components of the pillar tilting, with an error of ± 3 mm. (b) Tie rod tension in N/mm^2 and displacements (magnified 20 times) due to pillar tilting. (Anzani 2007).

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The traditional systems can be very efficient in the case of regular vertical or horizontal elements. In the case of vaults, domes, etc... which are curve elements, the type of survey which are more efficient are the ones which can survey and represent 3D models, as photogrammetric surveys and rather recently laser scanners.

3.1.3.2 Photogrammetry

The traditional photogrammetry is a survey based on stereo pictures made by two cameras mounted on a fixed support and taking pictures from a fixed distance. The restitution of the images is obtained by the use of a photogrammetric software. The photogrammetric survey can be aerial or from the ground.

In the following an example of a 3D reconstruction made by using normal digital cameras and a cheap software applied to a very irregular geometrical shape, a Temple in Mỹ Sơn (Vietnam), which is an archaeological area damaged by the Vietnam war, (Barazzetti 2009a,d).

The 3-D reconstruction of Mỹ Sơn G1 Temple represents a fundamental task in the restoration design process. Two main techniques can be applied today to this aim, also integrated between them: digital photogrammetry and terrestrial laser scanning (TLS). A detailed comparison of their capabilities and drawbacks can be found in (Grussenmeyer 2008).

In the case discussed here, the selection fell on the so called *soft-copy photogrammetry,* (Luhmann 2006), which adopts consumer digital cameras and low-cost commercial softwares based on *mono-plotting. Mono-plotting* is opposite to *stereo-plotting*, which requires a device for stereo vision that would increase hardware cost. This solution is low-cost, considering the whole hardware and software equipment (included PC and screen) can be purchased fot less than 5,000 Euros. On the other hand, terrestrial laser scanners and related data processing softwares are much more expensive (at least 15-20 times the cost of photogrammetry); furthermore, these are still cumbersome, resulting in problems in carrying them on the site of the survey.

But the analysis of the 3D reconstruction process cannot be limited to instruments for data acquisition only, as often is thought. In Figure 3.11the full pipeline of the 3D reconstruction and modelling processes based on both photogrammetry and TLS is reported. The background colour of each task visually highlights its complexity. In particular, here the pipeline of photogrammetry is discussed and documented by the results obtained from the survey of G1 Temple. More details about the use of photogrammetry in this project are given in (Barazzetti 2009a,d).

	Sensor calibration	Data acquisition	Sensor orientation	3D point-cloud reconstruction	Extraction of a vector model
Photogrammetry	Always required before a survey. Alternatively: self-calibration during image orientation, if there are enough images and targets.	Accurate planning of the survey to control: image scale, block geometry, object coverage.	Automation is still a not consolidated task in commercial SWs, unless targeting is used. Otherwise, it requires manual measurements.	Automation by dense matching, if there is enough texture on the images.	Manually (directly from oriented images).
TLS	TLS to be periodically calibrated in laboratory. Calibration of the integrated camera as in photogrammetry.	TLS has the same requirements than photogrammetry, but less critical to be worked out.	Automatic if the object presents irregular surface. Otherwise, targets must be used.	Automatic (TLS directly acquires point-clouds).	Very involved, unless integrated images are used to texture the point-cloud.

Figure 3.11 Pipelines for 3D reconstruction based on photogrammetry and terrestrial laser scanning; background colours highlight the complexity of each task: easy (green), medium (yellow) difficult (red), (Barazzetti 2009d).

Data acquisition was performed by two Nikon cameras: a D100 (6.0Mpx) with a 18.0mm lens, and

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a D80 (10 Mpx) with a 24.0mm lens. Calibration was carried out on-site before each data acquisition session by using Photomodeler 6 software, (www.photomodeler.com), and the calibration sheet included in the package. The practical survey was accomplished by people without experienced in such works, and consequently a simple block of images made up of a circular sequence around the building was designed. Furthermore, some coded targets were applied to the external walls to help *image orientation*, while total station measurements of some ground control points were already available. This geometry of the photogrammetric block gave out a good orientation of the images with both manual (by Photomodeler 6) and automatic procedures, (**Errore. L'origine riferimento non è stata trovata.**). On the other hand, many details could not be imaged due to occlusions, or they were captured at a too low scale for reconstructing the fine details as well (e.g. the shape of bricks). Furthermore, *coded targets* were in part not visible in images, preventing the use of the automatic orientation function of Photomodeler 6, which would allow to save time and man work. Indeed, automatic orientation was performed by a proprietary scientific software developed by the SITECH group at Politecnico di Milano, which does not require any targets.

In order to suggest some guidelines to best practises in future applications, we reckon that a simple image block geometry with large overlaps between convergent images is a viable approach to deal with many situations, (Luhmann 2006) and (Kraus 1997). However, such a block needs to be completed by further images aiming at details and covering occluded areas. Today the most low-cost photogrammetric packages implement procedures for automatic image orientation based on *coded targets*, as mention above. This solution allows also people with a low photogrammetric background to perform this task in short time. Otherwise, the orientation can be carried out manually through the measurement of corresponding natural points on different images. Thus putting coded marks on the object to be surveyed is still strongly suggested, while in a few years marker-less procedures will become more operational at the practitioners' level.

While TLS directly acquires a dense point-cloud describing in a discrete way the surface of an object, photogrammetry requires extracting the 3D information from the oriented images. This issue deserves to be further discussed here, because when photogrammetry and scanning are applied outside their native communities, it is very often misunderstood. Disregarding the applied tool, a survey might give rise two main 3D products: *vector models* and/or *point-clouds*. The former describe the fundamental geometry of an object, which is fully exhaustive when this has a regular shape, i.e. it can be approximated by known mathematical functions. The latter make use of a dense set of unorganized points, (no relationships occur among them), that approximate the surface in a discrete way. While this approach is more suitable than the other one for artefacts presenting irregular shapes (e.g. archaeological ruins, sculptures, bas-relieves and the like), point-clouds are not adequate for objects showing strong break-lines. Indeed, a point-cloud is made up of points coming from a sampling process that is not targeted to acquire specific points, like e.g. in the vector model construction. Secondly, in many applications for restoration and documentation of Cultural Heritage vector models are required, while point-clouds can be useful for 3-D visualization purposes and in Virtual Reality, to extract cross-sections (even though these require a post processing work (Berto 2001), (Berto L. et al. 2001), (Berto 2002), (Bianic 2001) and (Bienert 2008)) or as intermediate step to other products (e.g. orthoimages). This consideration will be also supported by comments reported in Section 4 on the results obtained at Mỹ Sơn Temple.

Now, photogrammetry does not directly give either vector models or point-clouds, but these can be then derived from images. The former by manually measuring points describing a feature in at least two images, (if mono-plotting is adopted), or on the stereoscopic model, (if stereo-photogrammetry is used). The latter can be performed by using dense matching techniques like those implemented in the "Scanner" module of Photomodeler 6, or another commercial or scientific packages (Barazzetti 2009b). In Figure 3.12, a 3D vector model and a point-cloud of the "G1" Temple are shown. On the other hand, TLS directly acquires a dense point-cloud, from which it is however very involved to extract a vector model. To do this, images are still needed for localization of break-lines, task that is frequently more complex to be performed in a 3D environment than in imagery. In many cases, photogrammetry has to be chosen, also motivated by the cheaper cost. On the contrary, when an object features an irregular shape, using TLS is better.
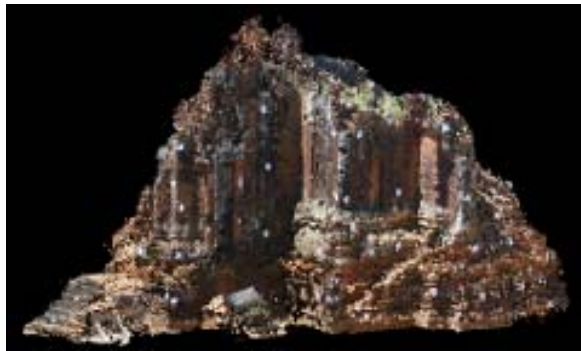
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Figure 3.12 On the left, a 3D vector model of the Mỹ Sơn "G1" Temple; on the right a 3D point-cloud of the same building. Both products were constructed through the photogrammetric process, (Barazzetti 2009d).

The pipeline described so far might give out other products, in addition to these already discussed: photorealistic 3D models, orthoimages, and anaglyph images for 3D vision. Traditional horizontal plans, prospects and cross-sections can be derived from vector models and point-clouds, if enough resolution was achieved.

Eventually, the 3D reconstruction process results in products that are not directly useful for 3D data management and analysis, because they are based on simple primitives and not on 3D shapes. This important task can be achieved only after 3D modelling, i.e. the construction of a solid model where the full object is modelled by 3D geometric shapes connected among them. The availability of a 3D model allows performing 3D structural analysis, (Taliercio 2007), to integrate or replace new portions of a building, to analyse deformations, etc... Unfortunately, the 3D modelling process is still a time-consuming task, to be performed by experts in a graphic environment where the photogrammetric model has to be imported. Practically, an operator has to identify the elementary blocks, to model them with regular shapes, and to apply a proper texture to them.

3.1.3.3 Laser scanning

Laser Scanner (LS) which is being adopted as a powerful tool to create 3D models of historical buildings and structures has become a sort of fashion for the geometrical survey of complicated architecture and volumes in the last years, (Brumana 2010).

Even if this tool is really powerful some comments are necessary in order to use it in a better and useful way.

LS allows the acquisition of enormous number of points (clouds) in few minutes, with high accuracy, thus it has been erroneously deducted a coincidence of the point clouds with the 'concept model'. The expected result is to have an immediate model, in the sense of a result which doesn't need any intermediated role of the scientific competences/context analysis/knowledge/geometric synthesis/historic background. The correspondence between surveyed points and the interpretation model is cancelled by this wrong conviction. The richness of information carried out by each point need to be interpreted by refining investigation, decoding instruments and representation tools, (Rinaudo 2007). As the Information Society has pointed out in these year, too much data can mean 'no data', if interpretation methods don't evolve in order to avoid losing information: the more we want to avoid softcopy reverse engineering of the reality, the more we need to strengthen conceptual schema in reading the building through interdisciplinary connection and representation methodology to manage multiple input within virtual space-time model. In addition, LS survey doesn't catch the points that exact belong to a profile, to the discontinuities, to the edges, to all those points we would like to obtain to simplify the reality. In fact the surveyed points are only function of a range angle (H,V) set up at the beginning of each surveyed clouds. Consequently the surveyed point doesn't immediately represent the schematized reality. We expect they cover an area 'around' the exactly position we would like to determine: consequently we cannot connect them directly without 'interpreting', selecting and abstracting the

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model to be reconstructed. In the case study 4 (Annex 1) a paper is attached which shows and application of the LS survey and how its use need a multidisciplinary collaboration.

3.2   DETAILED SURVEY OF MACRO-ELEMENTS

3.2.1   Survey of decay and damage of walls

3.2.1.1 Image rectification and ortho-images

When internal and external walls are made of facing bricks and stones and rendering is not present, the presence of old collapse and of partial reconstructions after various earthquakes can be surveyed by pictures taken with digital cameras, joined together and rectified or by ortho-images, from which it is possible to see the evolution of the wall. This was the case of the internal prospect of the central nave of the Syracuse Cathedral (Sicily), (Figure 3.13). It is possible to define through the different textures of the masonry the evolution of the walls after partial collapses due to earthquakes.
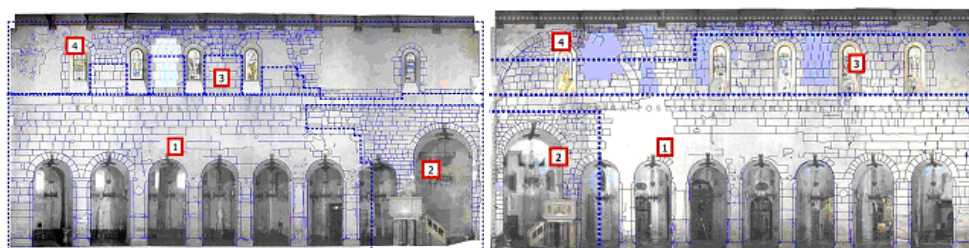
Figure 3.13 Identification of the main masonry textures and of the building phases - Syracuse Cathedral, Italy. (Binda 2007b).

3.2.1.2 Detailed rectified pictures and stratigraphy

When surveying vulnerability of buildings, also the surface decay and damage, the physical - chemical damage of the materials, the wrong repairs, etc..., can be vulnerable elements. Discontinuities due to the several evolution phases could lead to cracks or local stiffness changes. Therefore a detailed rectified photographic survey or a manual survey and stratigraphic lecture can help the designer. This was the case of a stable of a rural building, Cascina Chiesa Rossa in Milan, (Figure 3.14).

Figure 3.14 Rectified picture and detailed restitution of the damaged wall of a stable in a rural building in Milan. Source POLIMI.

3.2.2   Survey of connections

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The connection wall to wall can only be seen by direct inspection, (Figure 3.15 a); then the connection is visually investigated and a detailed drawing of it can be made accompanied by a written description, (Figure 3.15 b).

(a)   (b)
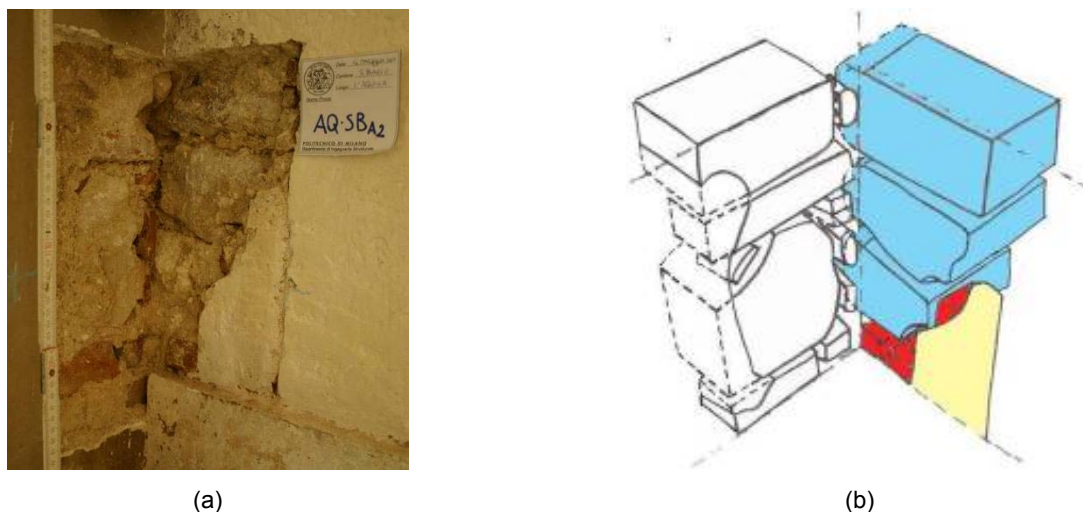
Figure 3.15 (a) Survey of a wall to wall connection and (b) drawing of the connection visually observed. Source POLIMI.

However, in many cases the characteristics of connections are hardly detectable due to presence of plaster or other decorative elements and lack of damage. In such situation, NDTs (thermovision, radar inspections, etc...) could be used within an addressed investigation project.

The same can be said of reinforcement: the initial survey should indeed aim to locate and describe any additional strengthening systems, but these may be embedded in the original material to the point of not being visible but only detectable by radar tests. Moreover, even in case of well visible reinforcement elements, like cross-ties, it may be difficult to verify that the reinforcement is well maintained and properly connected. In fact, the presence of damage directly related to lack of good connections or, conversely, the lack of specific crack patterns can provide indication of the quality and level of maintenance of the connections and of the strengthening system. Nevertheless, the initial visual survey should aim to report with as much detail as possible the position, number, typology, dimensions, number and quality of visible reinforcing elements.

Due to the uncertainties connected to limits of visual inspection, this is normally followed by an in-depth investigation performed by non destructive techniques that aim to:

- Assess the level of connection between two structural elements, (e.g. lay-out of brickwork/ stonework at the corner of two walls);
- Detect the presence of voids or detachment within the connection;
- Detect the presence of reinforcement elements.

Such investigation can be performed by any of the NDT described in the following sections.

3.2.3 Thermography for the geometrical survey

When the walls are covered by rendering and plaster and no information is given of the texture and type of masonry or in the case of vaults covered by frescoes and paintings, especially in monumental buildings, the removal of the rendering and plaster is not possible, then non destructive techniques are useful in order to see beyond the covered thickness. The best non destructive technique in these cases is without doubt thermography, (see the Sect. below).

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Figure 3.16 shows the texture of a pillar: the dark parts in the prospect are brick courses, alternated with stone blocks (whitish).
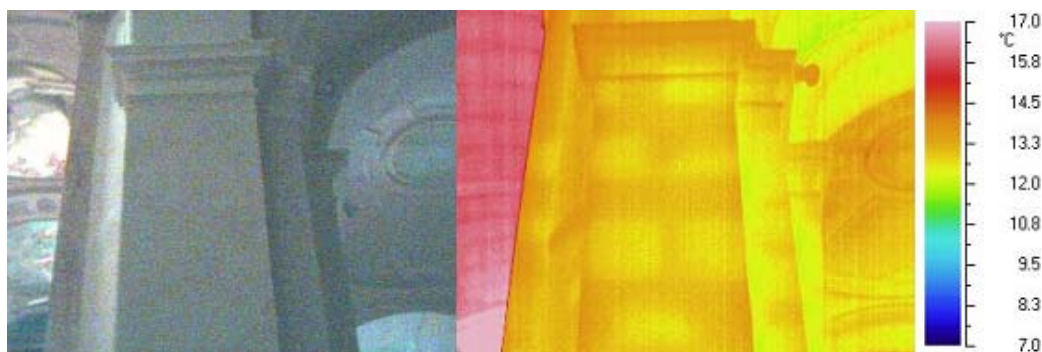
Figure 3.16 Thermographic survey of the masonry texture of a pillar made in stone and brick. Source POLIMI.

Figure 3.17 shows the structure of a timber vault hidden by a painted plaster. This survey was very important to study the behavior of the timber structure without any destructive investigation. In order to characterise the materials only small samples of the rendering were taken.
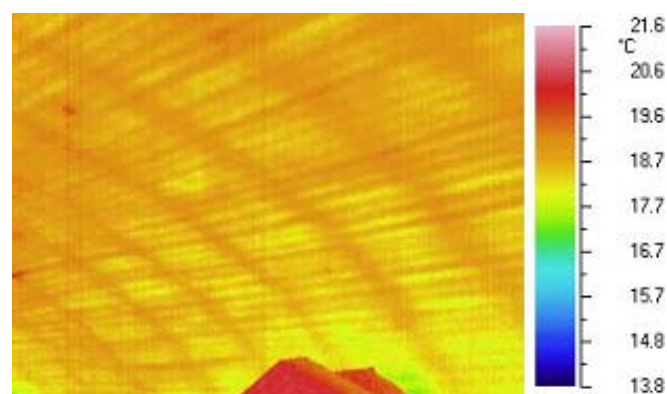
Figure 3.17 Thermographic image of a timber vault hidden by a painting. Source POLIMI.

3.3 HISTORICAL EVOLUTION

The analysis of the evolution of the building or aggregate along the time is a very important step in order to check the vulnerable elements which can cause structural faults or effective structural connections. The reconstruction of the historical evolution of the structure and of the subsequent volumes is essential to find the resisting structural system as a result of the continuous modifications.

The information can come from indirect sources of from direct sources. Indirect sources are the historical documents found in the archives, the subsequent maps from the municipality, the description of damages collected after the previous earthquakes, the old pictures, etc... Direct sources are all the investigation described above with special attention to the details which can indicate non homogeneous parts.

This phase is useful to reconstruct or obtain through various hypotheses the chronological succession of the constructive phases of the building or aggregate. In the case of e aggregates this evolution represented on plans, prospects and sections is useful to find possible discontinuities in the walls, floors and roofs and also the variability in the morphology of the masonry walls, (variability in texture or in the section: one leaf, two leaves or more).

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In Figure 3.18 the evolution of the Syracuse Cathedral in Sicily is presented from its origin. In Figure 3.19 the historical evolution of a church near Brescia damaged by the 2004 earthquake near the lake of Garda is also presented.
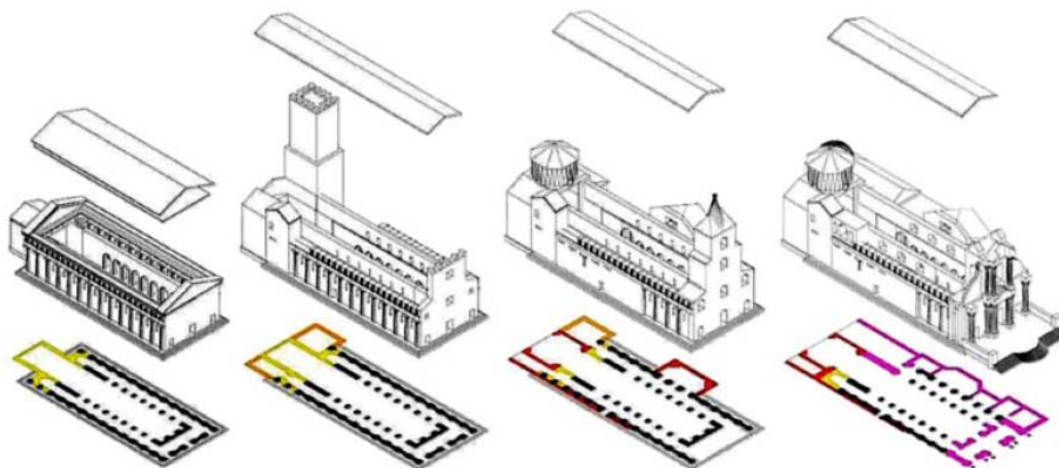
Figure 3.18 An example of axonometric projection of the Siracuse Cathedral evolution: the colours in the plan enhance the interventions before the earthquake of the 1169, 1542, 1693, 1800, (Binda 2007b).

(a) (b) (c) (d)
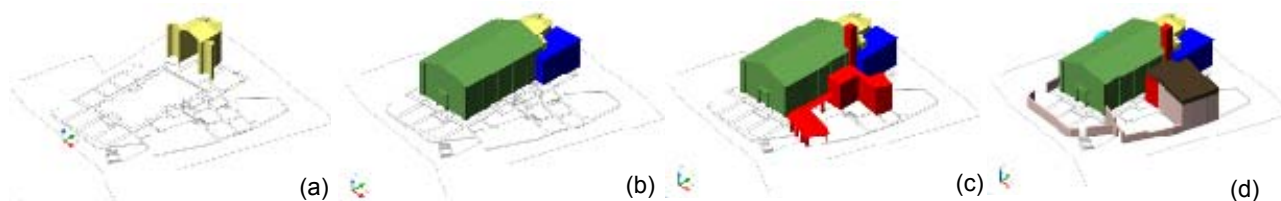
Figure 3.19 An exemple of axonometric projection of the St. Michael Church evolution at Sabbio Chiese (Italy): the colours in the plan enhance the historical phases: (a) high Middle Age century, (b) 15th-16th century, (c) Napoleonic period, (d) Early 20th century). (Binda 2006c).

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4 CRACK PATTERN AND SURVEY REPRESENTATION

4.1 METHODOLOGY

The crack pattern survey is very important to define eventual critical situations due to defects in the structural response to static ordinary actions or to seismic actions, (Binda 2010a).

The presence of pre-existing defects or cracks due to static actions, (soil settlements, out of plumb, high compression, local settlements, etc...) represents an element of vulnerability in case of seismic actions which can produce specific collapse mechanisms.

Also physical-chemical decay of the materials, (stones, bricks, mortars, timber elements, tie rods, etc...) represents a potential cause of vulnerability in case of seismic events, which can produce local damages in the weakest points: in this special cases the survey can be made through maps of the damage distribution together with the crack pattern survey.

When the structural damages are due to the seismic action, the crack pattern survey associated to the critical geometrical survey of the building or aggregate, will allow formulating the first emergency qualitative hypotheses and interpretations on the causes of the mechanisms of damage or collapse.

The crack pattern has to be surveyed and represented with reference to the structural elements which can be identified as macro-elements of the structure, (walls, floors, vaults, roofs), (Binda 2010a). The cracks will be classified according to their direction (vertical, diagonal, horizontal, curved, etc...), (Mastrodicasa 1993); they will be reported according to their dimension (extension, width, passing through the wall, non-passing through, etc...) on the existing plan, prospects and sections. Other typologies of damage as detachments, rotations, sliding, out-of-plane displacements of the various structural elements have also to be represented.

A first reading and representation of the structural damages can give a synthetic description of its type and of its causes and also a first qualitative evaluation, (GNDT 1999), (AEDES 2000) and (PCM 2000, 2001). This operation is especially important during the emergency after an earthquake, but also after other events like floods and fire.

4.2 ANALYSIS OF HISTORIC DAMAGES, OF PREVIOUS REPAIR INTERVENTIONS AND OF THEIR EFFECTS

The interpretation of the damage causes, on the basis of the survey described above, should be anticipated by a recognition of eventual previous damages documented for the building or aggregate, due to previous earthquakes or to static actions.

Also it is important to survey and document eventual strengthening or repair interventions carried out after previous earthquakes or to repair existing damages, in order to check their effectiveness. The documentation can be found at the Municipalities or at other offices which have eventually collected it in the past. This information is very important to compare the pre-existing damage to the present one after a seismic event.

4.3 CRACK PATTERN INTERPRETATION

The methodology suggested above gives the basic elements to arrive to a critical examination of the visible crack pattern in order to formulate hypotheses on the nature and typology of the structural damage. Therefore, the hypotheses on the (global or local) damage mechanisms activated in the building or in the aggregate and their potential causes should be qualitative and preliminary, (Binda 2010a).

Figure 4.1 shows the case of a highly damaged tower in Italy. The reading and interpretation of this crack pattern was the base for decisions on the further diagnostic investigation and on the first provisional interventions, (Anzani 2008).

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Figure 4.1 Crack pattern survey of the internal prospects of a highly damaged tower in Biella (Italy), (Anzani 2008).

The "critical" mechanism of collapse can be defined as the mechanism which among all the kinematic possible ones, has allowed the initiation or the collapse of the structure. In practical terms the constraint conditions are given by the distribution and by the typology of the connections, strengthening or contrast details, (ties, buttresses, tie beams, thicker parts of the walls, etc...), which can oppose to the elementary damage mechanisms, nevertheless they can leave some free displacements of the structures to secondary modes of damage which will be shown by the earthquake.

In synthesis the detailed comparison between the overall structural system, the crack pattern of the occurred damage, the presence of constraints should allow the qualitative individuation of one collapse mechanism among all the possible ones, (Binda 2007c,d).

The quantitative analysis of the formulated hypotheses comes subsequently through mathematical modelling and evaluations of the structural safety, so that it is possible to make compatible choices for the intervention, (Munari, 2009) and (Valluzzi 2007).

4.4   CRACK PATTERN SURVEY DURING EMERGENCY

The Italian Department of Civil Protection and Ministry of Cultural Heritage had in fact prepared soon after the Umbria Marche earthquake a special template to be filled on-site during the emergency phase by special teams composed by structural engineers, architects, experts of Cultural Heritage Department, Civil Protection Department and firemen. This template was to be used for the collection of data on churches. In the following, just some months before, a second template on Palaces was also published which was used for the first time after the L'Aquila earthquake (Annex 2), (GNDT 1999), (AEDES 2000) and (PCM 2000, 2001, 2005).

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5 ON-SITE ASSESSMENT OF MASONRY STRUCTURES

5.1 NON DESTRUCTIVE TECHNIQUES

The importance of evaluating existing masonry buildings by non-destructive investigation carried out on-site has been mentioned by many authors, (Binda 2000). NDTs can be used for several purposes: (i) detection of hidden structural elements, like floor structures, arches, piers, etc..., (ii) qualification of masonry and of its composing materials, mapping of non homogeneity of the materials used in the walls, (ex: use of different bricks in the history of the building), (iii) evaluation of the extent of mechanical damage in cracked structures, (iv) detection of the presence of voids and flaws, (v) evaluation of moisture content and capillary rise, (vi) detection of surface decay, and (vii) evaluation of mortar and brick or stone mechanical and physical properties.

5.1.1 Thermovision

Thermovision is a NDT which has been applied since several years to works of art and monumental buildings. The thermographic survey has the advantage of being applicable to wide surfaces of walls; it is a telemetric method and has high thermal and spatial resolution.

The thermographic analysis is based on the thermal conductivity of a material and may be *passive* or *active.* On the *passive* application it is analysed the radiation of a surface during thermal cycles due to natural phenomena, (insulation and subsequent cooling). If the survey is *active*, forced heating to the surfaces analysed are applied.

The thermal radiation is collected by a camera sensitive to infrared radiation. In fact each material emits energy, (electromagnetic radiation), in this field of radiation; this radiation is characterised by a thermal conductivity, which is the capacity of the material itself of transmitting heat, and its own specific heat.

Each component of an inhomogeneous material like masonry shows different temperature. The thermovision detects the infrared radiation emitted by the wall. The result is a thermographic image in a coloured or B/W scale. At each tone corresponds a temperature range. Usually the differences of temperatures are fraction of degree.

The total flux of energy E emitted by a surface, is the sum of the energy E_c emitted by the surface by thermal excitation and the flux E_r that is emitted by the surface around each point.

$$E = E_c + E_r$$

Eq. (5.1)

The infrared camera, measures the energy flux E. The tests are carried out at a certain distance without any physical contact with the surface.

Active thermovision can be also carried out to tests more on depth. The surface of the tested wall should be heated for a certain time, this way the thermal conductivity of the internal part of the masonry is shown up to a certain depth. The infrared camera transforms the thermal radiation in electric signals, successively converted in images. These images can be visualised on a monitor and recorded on a computer. In the video camera, the infrared radiation that reaches the objective is transmitted by an optical system to a semi-conductor element. This last one converts the radiation in a video signal, while the surveying unit signal processes the video camera signals and shows the thermographic image, (Figure 5.1).

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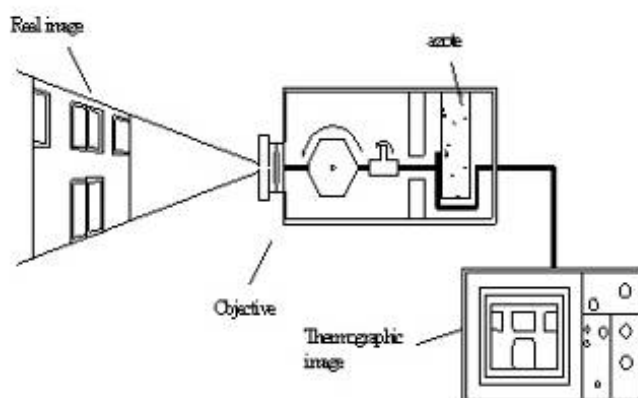
Figure 5.1 Scheme of the thermovision camera, (Bellini 1986).

Thermovision can be very useful in diagnostic; in fact it is used to identify areas under renderings and plasters that can hide construction anomalies. It is particularly interesting for studies on frescoed walls, where it is not possible to take samples or use testing techniques that come in contact with the frescoed surfaces, (Figure 3.16 and Figure 3.17). Other applications can be: (i) survey of cavities, (ii) detection of inclusions of different materials, (Figure 5.19), (iii) detection of water and heating systems, (iv) moisture presence. In the presence of moisture, the camera will find the coldest surface areas, where there is continuous evaporation. The evaporation is due to the difference in R.H. between the inside of the masonry and the environment outside and to natural air movements.

Figure 5.2 and Figure 5.3 show as an example the use of thermography to detect the details and the presence of tie rods hidden by the rendering in the porch of a palace in Milan (Italy), (Binda 2003b).
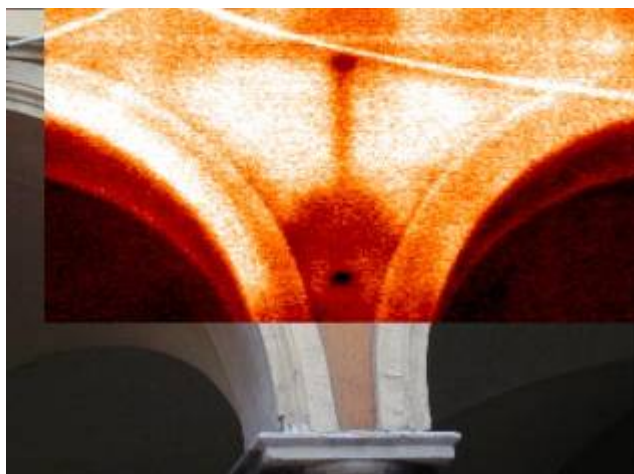
Figure 5.2 Thermovision applied to detect a vertical rod behind the plaster, (Binda 2003b).

Figure 5.3 Data of Figure 5.2 after the application of a gradient filter to enhance the target, (Binda 2003b)

Figure 5.4 shows the use of thermography to detect the rate of drying of a wall after a simulated flood, (Binda 2010b).

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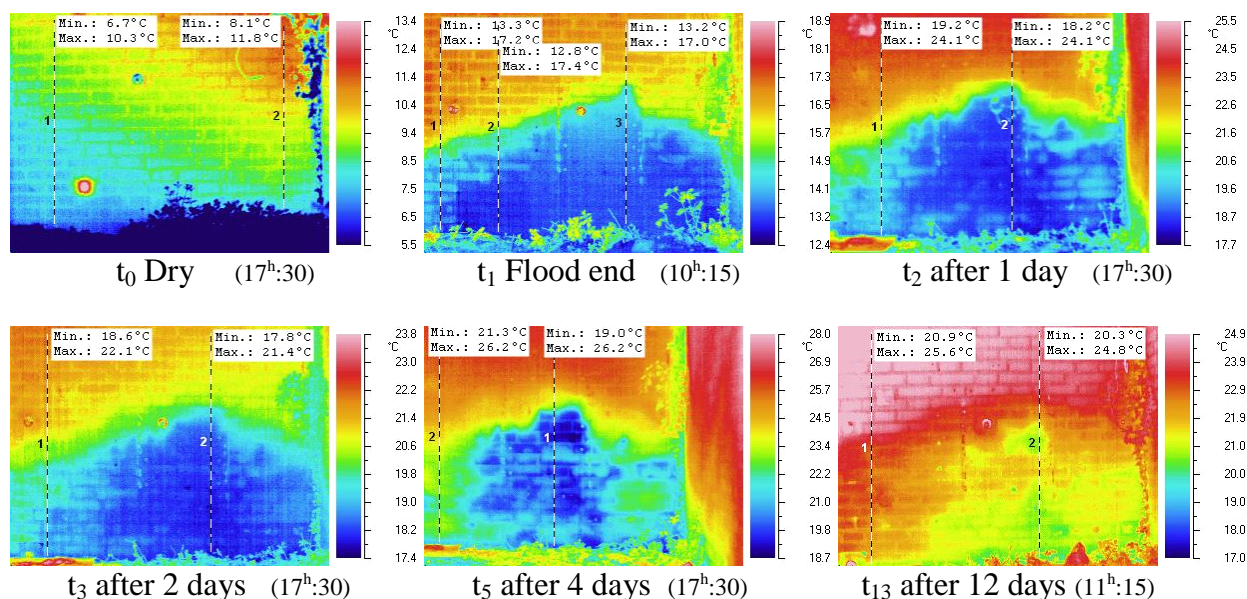
Figure 5.4 Passive thermography of a wall from the condition flooded to drying after 12 days (red color means dry, blue color means wet in the scale), (Binda 2010b).

In the diagnosis of old masonries, thermovision allows the analysis of the more superficial leaves, in absence of a thermal irradiation, (Figure 3.16 and Figure 3.17). It is necessary to point out that the penetration depth of this technique is limited, so it is unable to locate anomalies which are hidden in the inner part of the masonry. The technique is often sensible to the boundary condition of the tests. Sometimes shapes are detected, caused by different local emissions and not by effective variations.

The interpretation of thermographic data can be more accurate when appropriate software is available, (Figure 5.19). Nevertheless it is necessary to focus the mathematical model on the specific problem to obtain specific indications on how infrared images are to be recorded, (Lenzi 1997), (Binda 2003b). This could be used as an effective mean for the analysis of heat transfer through building material. By using this model it is possible to study the energy balance of the wall surface, and to measure the temperature of the surfaces where transpiration occurs. The amount of transpiration is affected by: the air and wall temperature, the radiant heat, the wind speed and the relative humidity of the air inside the pores of the materials.

5.1.2   Sonic pulse velocity

Among the ND investigation methods, the sonic methods are with no doubt, the most widely used. The testing technique is based on the generation of sonic or ultrasonic impulses at a point of the structure. A signal is generated by percussion or by an electrodynamics or pneumatic device, (transmitter), and collected through a receiver which can be placed in various positions, (Figure 5.5).

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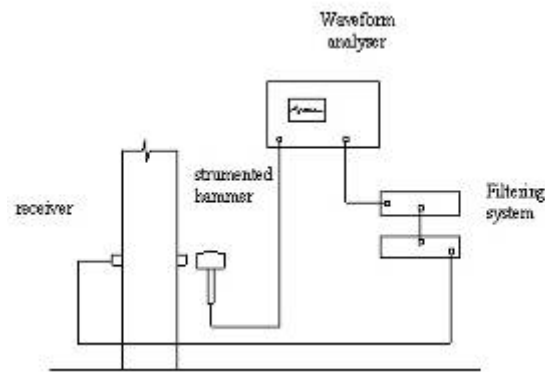
Figure 5.5 Scheme of the sonic test equipment, (Bellini 1986).

The elaboration of the data consists in measuring the time the impulse takes to cover the distance between the transmitter and the receiver, (Binda 1997c). The use of sonic tests for the evaluation of masonry structures has the following aims:

- to qualify masonry through the morphology of the wall section (Figure 5.6), (Binda 2009c);
- to detect the presence of voids and flaws and to find crack and damage patterns;
- to control the effectiveness of repair by injection technique and others which can change the physical characteristics of materials, (Binda 1993), (Binda 2001).

The first applications of sonic tests to the evaluation of masonry materials and structures have been carried out on long time ago in the sixties, (Aerojet General Corporation 1967). The difficulty of interpretation of the results in the case of inhomogeneous materials like masonry was always known and the first results were clearly interpreted as qualifying rather than quantifying values. Several efforts have been put in the tentative of interpretation of the data from sonic and ultrasonic tests, (Abbaneo 1995), (Colla 1997).

The limitation given by ultrasonic tests in the case of very inhomogeneous material made the sonic pulse velocity tests more appealing for masonry. Nevertheless, in the case of low porosity units and mortar used in solid or cavity walls, ultrasonic tests can also be successfully used. Efforts have been made by the authors to correlate the sonic parameter to the mechanical characteristics of the material, but this correlation seems difficult, (Abbaneo 1996), (Binda 2007a). Nevertheless after the tests, a high number of data are collected which are certainly meaningful not only for the local situation but also for the overall state of the wall.

The limits of sonic tests to masonry can be defined as follows:

- cost of the operations due to the high number of measurements which has to be carried out;
- difficult elaboration of the results due to the difficulties created by the inhomogeneity of the material;
- need for the calibration of the values to the different types of masonry.

The fundaments of wave propagation through solids allows to recognise the theoretical capabilities and limitations of the technique. The velocity of a stress wave passing through a solid material is proportional to the density ρ, dynamic modulus E, and Poisson's ratio v of the material. Hence, only global variations in these three parameters could be indicated. Resolution in terms of the smallest recognisable features is related to the dominant wave-length, (as determinate by the frequency), of the incident wave and also to the size of the tested element.

Wave-length, λ, is determinate by a simple relationship between velocity, v and frequency f:

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$$\lambda = v/f \qquad \text{Eq. (5.2)}$$

Hence for a given velocity as the frequency increases the wave length decreases, providing the possibility for greater resolution in the final velocity reconstruction. It is beneficial, therefore to use a high frequency to provide for the highest possible resolution. However there is also a relationship between frequency and attenuation of waveform energy. As frequency increases the rate of waveform attenuation also increases limiting the size of the wall section which can be investigated. The optimal frequency is chosen considering attenuation and resolution requirements to obtain a reasonable combination of the two limiting parameters.

In general it is preferable to use sonic pulse with an input of 3.5 kHz for masonry. Ultrasonic are not indicated in the case of highly inhomogeneous masonry as multiple-leaf walls.

Mechanical pulse velocity equipment can be used to acquire pulse velocity data. The input signals are generated by a hammer, often instrumented, and the transmitted pulse is received by an accelerometer positioned on the masonry surface. Some other instruments generate impulses by means of a pendulum apparatus, which permits repeatable input waves. In this last case the hammers provide a mass falling down from the same distances, (Abbaneo 1996). The frequency and energy content of the input pulse are governed by the characteristics of the hammer, (Suprenant 1994). Small metal plates can be glued to the masonry surface with a rigid epoxy resin to act as points of application of the receiving accelerometer. Signals are stored by a waveform analyser coupled with a computer for further processing.

Three types of tests can be conducted: (1) direct (or through-wall) tests in which the hammer and accelerometers are placed in line on opposite sides of the masonry element, (2) semi-direct tests in which the hammer and accelerometers are placed at a certain angle to each other, and (3) indirect tests in which the hammer and accelerometer are both located on the same face of the wall in a vertical or horizontal line.

The velocity and waveform of stress waves generated by mechanical impacts can be affected by:

- Input frequency generated by different types of instrumented hammers and transducers;
- Number of mortar joints crossed from the source to the receiver location - the velocity tends to decrease with the number of joints;
- Local and overall influence of cracks;
- Input frequency changes with the characteristics of the superficial material, (ex: presence of thick plaster or cracks). The sonic test in this case shows a very important limit. Due to the wall structure or to the presence of a thick plaster, (with fresco), the high frequency components could be filtered. The output signals have a rather low frequency content. Since the wavelength is equal to the ratio between wave velocity and frequency, this effect led to an output signal that contains only very long wavelengths. The sonic test in this case does not have a resolution able to detect in detail the wall morphology but it gives an overall description of the position of low velocity points.

Figure 5.6, (Binda 2010d), shows the maps of the velocities measured by sonic pulse velocity tests by transparency through the section of two pillars of a church damaged by the L'Aquila earthquake, in two perpendicular directions. Red colour means high velocity in the reported scales while blue colour means low velocity, (hence lower density). Figure 5.6a represents the points and directions of measurement, Figure 5.6b the map of a pillar section of uniform rather high density, Figure 5.6c the map of a damaged pillar with a weaker material inside the section.

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(a) (b) (c)
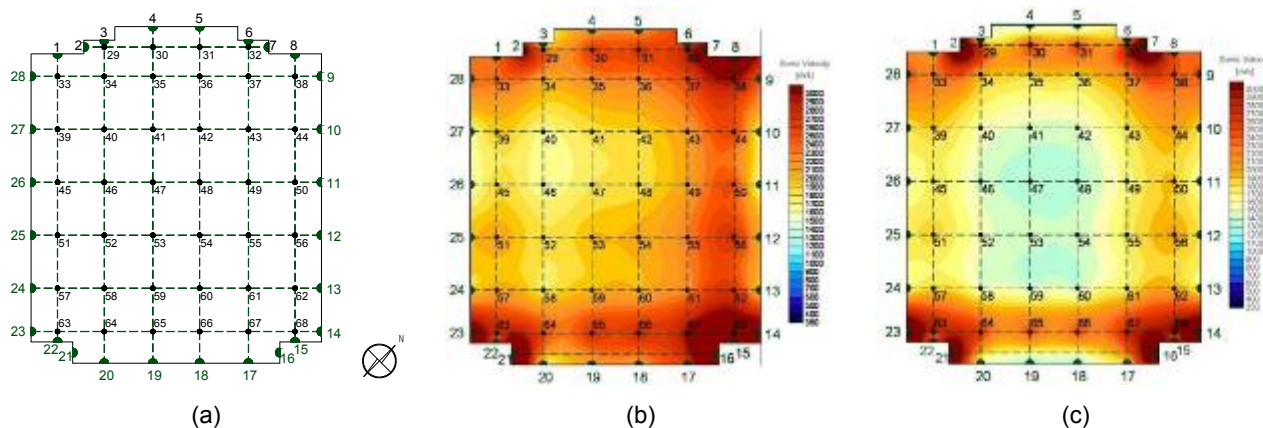
Figure 5.6 Maps of sonic velocities along the horizontal section of two pillars of a church in L'Aquila, (Binda 2010d).

Figure 5.7 shows the results of sonic tests applied in the case of injection by grouting to the base of one of the collapsed piers of Noto Cathedral, (Binda 2000, 2001). The increase in velocity means successful injection.
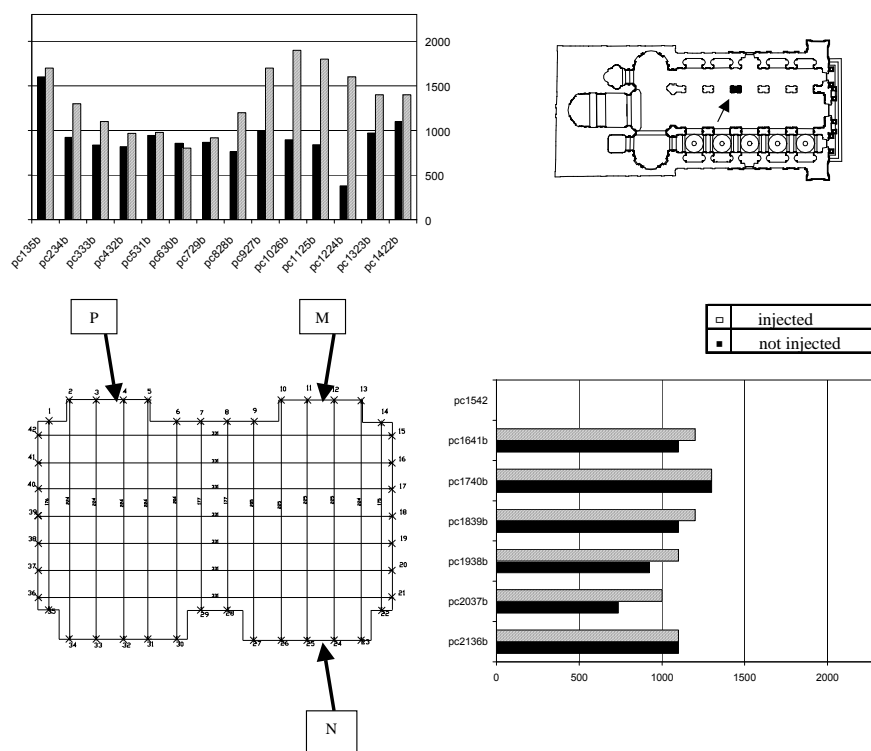
Figure 5.7 Noto Cathedral: results of sonic tests applied in the case of injection by grouting to one of the collapsed piers, (Binda 2000), (Binda 2001).

5.1.3 Ultrasonic

The method is based on the transmission and/or reflection of ultrasonic waves generated by an ultrasonic transducer or transducer array. The velocity of propagation depends on mechanical parameters of the structure, the reflection on the contrast of the acoustic impedances at the interface.

The principle of the ultrasonic echo technique with separated transmitter and receiver is based on the emission and reflection of impulses generated by a transducer. Inner voids in a specimen can

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be regarded as an interface between two different materials, (brick/air), for the propagation of sound and lead to total reflectance of the ultrasound waves. The propagation time of the reflection echo is proportional to the depth of the reflector (assuming a constant velocity of propagation).

The aims of the technique are: determination of the thickness of walls and single leaves, location of voids having sizes in the order of the wavelength of the ultrasonic waves (20.0 to 100.0mm) depending on frequency of the emitted pulses, characterisation of cracks (limited), correlation of ultrasonic velocity to compressive strength (limited).

Ultrasonic tests, carried out across cracks e.g. in case of pillars or columns, allow to estimate the depth of the cracks, (Bungey 1982). In the Cathedral of Syracuse, (Binda 2007b), ultrasonic equipment, (with 40.0kHz transducers), was used to estimate the depth of important cracks observed through the crack pattern survey.

Surface measurements were carried out. The emitting device was placed at the same distance of the receiving one, in respect to the crack, (Figure 5.8), (Bungey 1982). The tests, carried out on the pillars of the Syracuse Cathedral, showed that the cracks could have a great depth up to 40.0cm, (Binda 2007b).

Figure 5.10 shows an example of the crack's depth evaluation applied to a monolithic stone column, (ASTM 1991a).
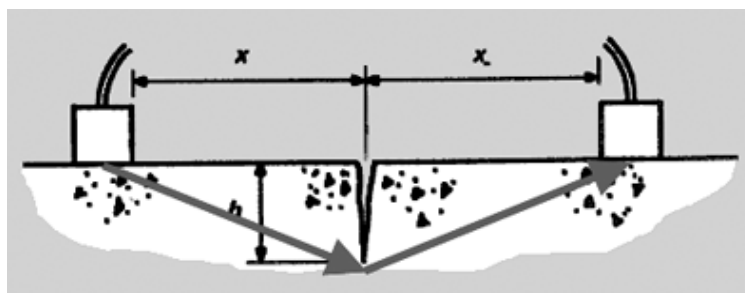
Figure 5.8 The emitting device placed at the same distance of the receiving one, respect to the crack, allows a rough estimation of the crack depth.

| (a) | (b) | (c) |
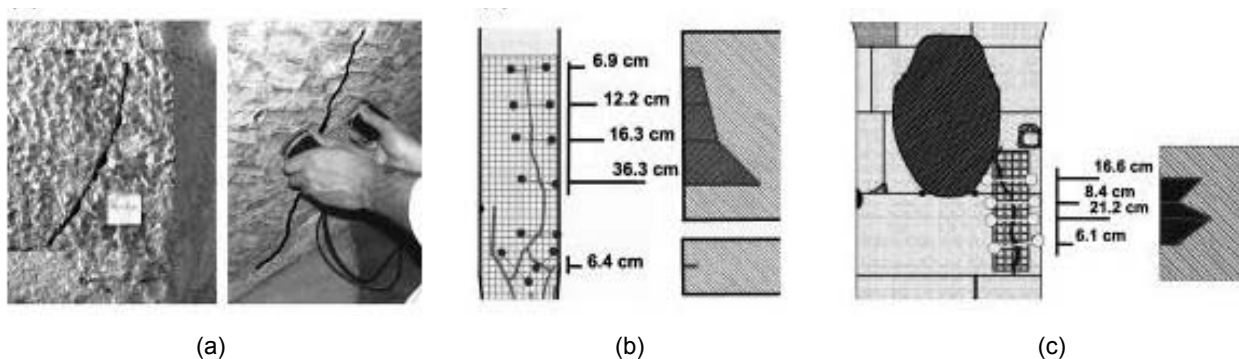
Figure 5.9 (a) Ultrasonic test procedure and (b) results of the tests on the pillars 31 and (c) 32, (Binda 2007b).

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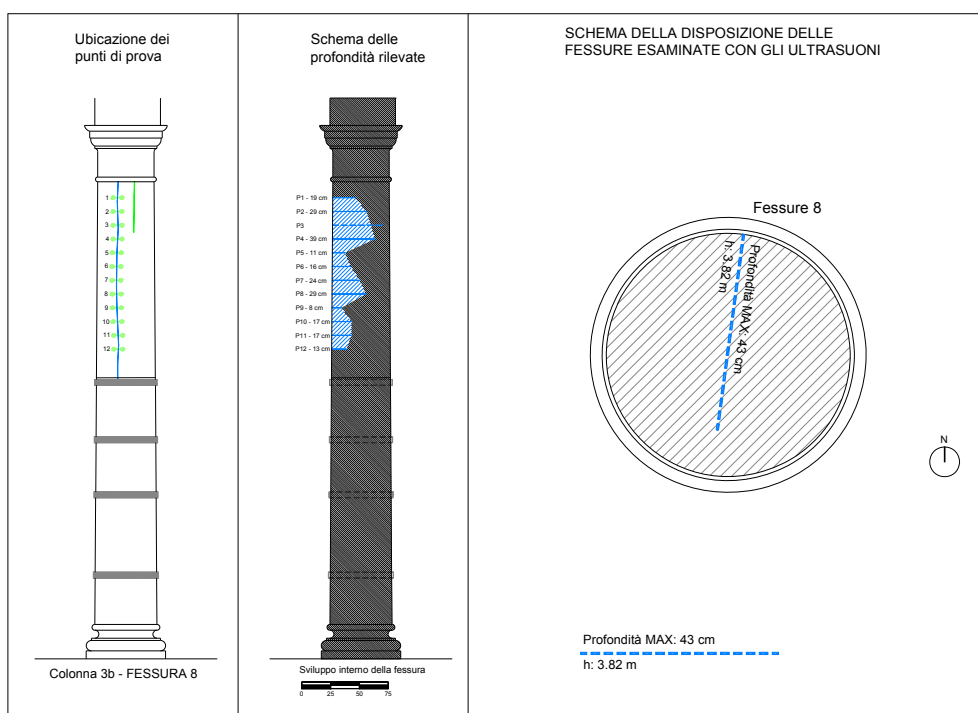
Figure 5.10 Evaluation of the crack's depth in a stone column. Source POLIMI.

5.1.4 Radar tests

Among the techniques and procedures of investigation which have been proposed in these last years, georadar seems from one hand to be the most promising, from the other to need a great deal of study and research. Being masonry an inhomogeneous material, all the techniques applicable to the homogeneous ones seem to fail unless an appropriate calibration is made.

The application of radar procedures are the following (Binda 1997b, 1998b, 2003b, 2009g):

- to locate the position of large voids and inclusions of different materials, like steel, wood, etc..., (Figure 5.11);
- to qualify the state of conservation or damage of the walls;
- to define the presence and the level of moisture, (Binda 1998a, 2010b);
- to detect the morphology of the wall section in multiple leaf stone and brick masonry structures, (Figure 5.12), (Binda 2008c, 2009c).

Georadar seems to be a powerful tool to detect the presence of voids and structural irregularities, the presence of moisture and hopefully the presence of multiple-leaves in stone masonry.

The method is based on the propagation of short electromagnetic impulses, which are transmitted into the building material using a dipole antenna. The impulses are reflected at interfaces between materials with different dielectric properties, i.e. at the surface and backside of walls, at detachments, voids, etc..., (Figure 5.13). When the transmitting and receiving antennas, which are often contained in the same housing, are moved along the surface of the object under investigation, radargrams, (colour or grey scale intensity charts giving the position of the antenna against the travel time), are produced. Measuring the time range between the emission of the wave and the echo, and knowing the velocity of propagation in the media it would be possible to know the depth of the obstacle in the wall. In the real cases, the velocity in unknown because it changes

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from one material to the other or in the presence of voids. Furthermore the velocity is higher in dry walls, and lower in wet walls.

(a)   (b)

Figure 5.11 Comparison between the results of the (a) radar and (b) termography, (Binda 2000).

Figure 5.12 Radargrams of the walls sections of Fontanella Church Bergamo, Italy. (Binda 2000).
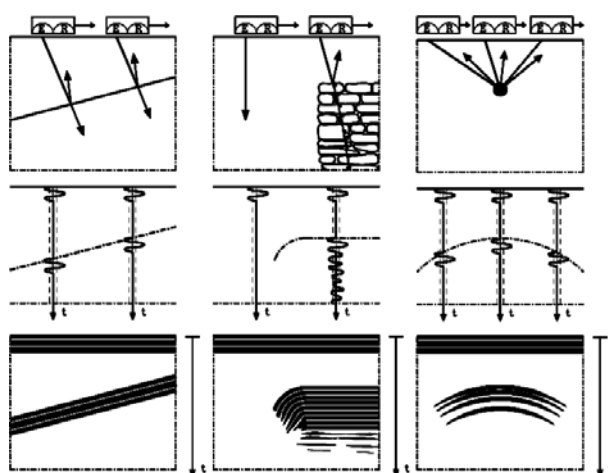
Figure 5.13 Example of features in the radar testing of masonry, (Kahle 1990).

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The interpretation of the radar data involves the identification of significant anomalies. It should be a recognition process detecting features on the records that are characteristic of known signatures. Identifiable features on a radar record are continuous reflection from laye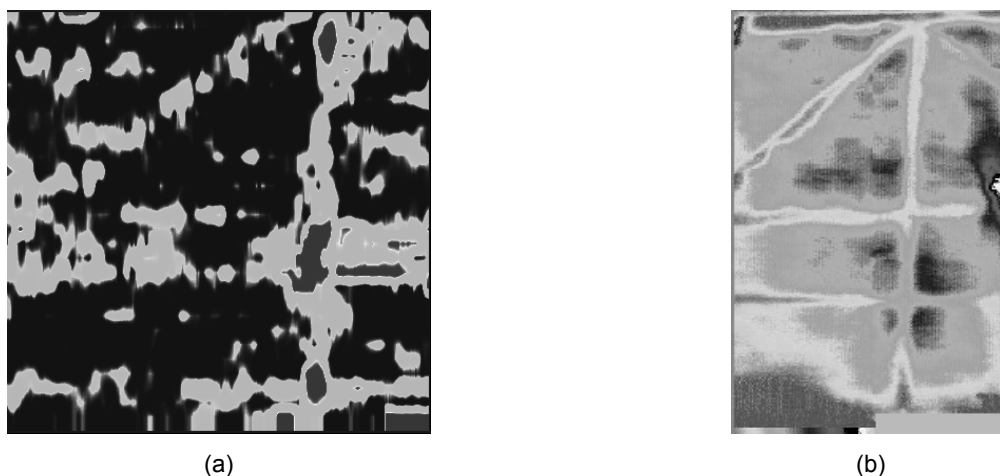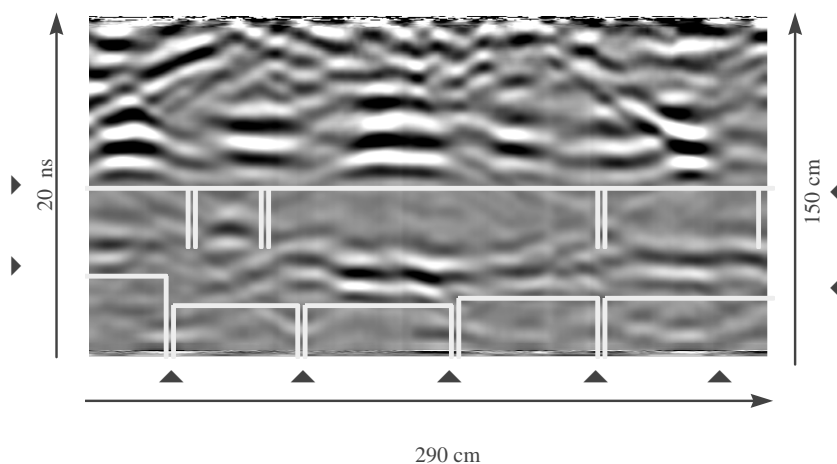rs or reflection from discontinuities like voids and local inhomogeneities in the masonry. A systematic approach was attempted by the authors with the aim of giving guidelines to the designers for an easy interpretation of the results. Special procedures have been studied to eliminate the typical interferences and hopefully to produce easily readable plots after elaboration of the test results. The main problems in the data interpretation are caused by disturbances in the signals due to the following causes:

1. Break-through effect. This effect is commonly visible on radar records and it hides partially the wall characteristics. It is caused by the fact that the antenna itself reacts to the electromagnetic wave.
2. Multiple echoes due to the presence of layers and joints.
3. Superposition of the lateral echoes that create images of parallel reflectors. This effect could be enhanced for a wall by the lack of mortar joints and the regularity of the stones.

Data reduction is generally limited to simple filtering operations to remove unwanted noises due to the data acquisition and to the equipment interference. The single signals are processed by: running average (high-cut filter), running median (low-cut filter), offset (zero-crossing trace alignment), etc...

Some trials have been done to remove the break through effects. First of all different antennas have been used. The use of higher frequency antennas could be the simplest method to solve the problem. E.g. in the tests carried out by the authors it was possible to observe that the 900.0MHz antenna has a long break through effect but it is able to detect the opposite side of the wall. On the contrary the 1.0GHz antenna has a short break through effect, (so it is possible to see clearly the first layer of the models), but it produces too weak waves, unable to reach the opposite surface, (Binda 1997c).

5.1.4.1 Experimental procedures and data processing.

Radar tests need always a preliminary calibration to verify the characteristics of the antenna in relation to the aim of the research. The test lines should be selected to be representative of the masonry and of the problem to investigate. The horizontal or vertical surveying-lines are usually located away from the wall edges in order to avoid boundary effects or away from other sources of surface noise (Figure 5.14). There are different type of acquisition in function of the aim of the tests. Usually echo modality is used, that is with the receiver and transmitter on the same side of the wall, moving the antenna along a surface (Figure 5.15 b). Other typologies e.g. by transmission could be useful for the tomographic acquisitions (Figure 5.15).

Figure 5.14 Geometry of acquisition, (Binda 2000).

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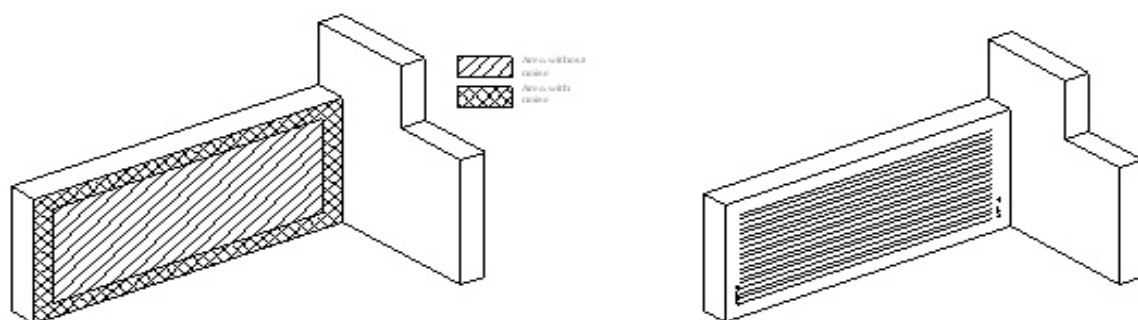HERITAGE FROM EARTHQUAKE-INDUCED RISK

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The first step is to carry out some measures in transmission, that is with the transmitter and receiver positioned on the two opposite sides of the wall, (Figure 5.15 d). It is so possible to verify if the emitted signal is powerful enough to detect the opposite side of the wall and the wave speed. This step allows to calibrate the relationship between the time and space scales, e.g. to localise anomalies like voids or layers in the depth of the wall. Some types of equipment give directly this transformation by setting up a value of the dielectric constant of the masonry. The value is an average of the characteristics of all the materials crossed by the wave.

A metal plate should be always positioned at the surface of the wall opposite to the emission surface. This procedure verifies that enough energy exists in the signal to allow the detection of the thickness of the wall, (Figure 5.16), and to control the data processing results. The tests are always carried out along 4 to 5 different parallel lines, every 5.0cm moving the antenna regularly without time delay/advance or vertical fluctuations. Reasonable values for the velocity of the antenna along survey lines are 5.0cm/s or 2.5cm/s. The previous operation is repeated rotating the antenna in 3 directions: horizontal, vertical and at 45.0°. The antenna signal is strongly polarised. The different orientation of the antenna allows for enhancing horizontal or vertical characteristics. It has been observed that the records with a 45.0° orientation of the antenna contain less local reflections and show clearly the general structure of the wall. This effect is due to the masonry characteristics with vertical and horizontal joints and to the emitted signal that is highly polarised. Furthermore the redundancy of the data is necessary for the data processing procedure.

The choice of the antenna must be made on a site basis. During the test it is important to control the radar potentialities in relation to the frequency used, (Padaratz 1995). These characteristics are not constant; they change for every test. The phenomenon is connected to the wall depth and to the presence of humidity; in fact it is known that the humidity decreases the wave velocity. For some particular cases the repetition of the echo-radar data acquisition on the other side of the wall along the opposite line could be necessary.
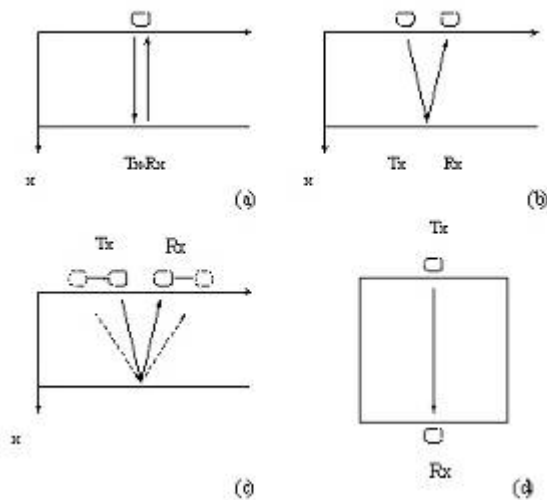
Figure 5.15 Modality of acquisition, (Binda 2000)   Figure 5.16 Effect of a metal plate, (Binda 2000).
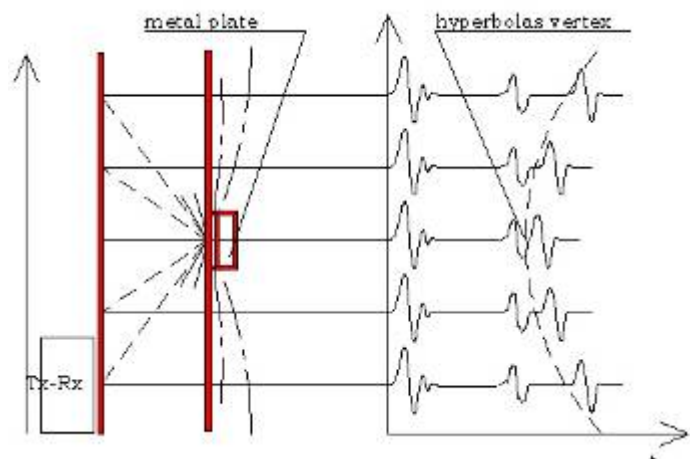
The following processing of the data is finalised to two main aims: (i) to clean the signals from general noise due to local echoes, (ii) to enhance the recognised features. The second aim tries to

increase the utility and reliability of the radar data. In fact usually radar data are clearly readable only by experts. It is important to show results, as radargrams and graphics, which are significant to operators like architects and engineers.

The first data processing on the single signal are filtering operations like running average and running median. The antenna signals decreases with the depth of the wall; these characteristics are usually less readable.

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An exponential gain technique is often used to amplify the signals ends. The plot of the derivative of the signals gives a better visualisation of the wave. The noise on this type of radar records comes from a variety of local irregularity in the masonry. The noise reduction is obtained by the sum or by the median of the signals of more radar traces recorded at a distance of 50.0mm and with different antenna orientations. The differences in the radar traces could be considered as secondary reflections and in this way are reduced. The noises due to local reflections occurs in different time positions. By summing each trace, of the radar section, the common and global coherent characteristics should be enhanced with respect to the noises.

A technique could be applied to point out local defects. The technique can be very efficient when the aim of the radar surveying is to detect local voids, defects or irregular structural elements. A significant signal of the radar section is subtracted from each signal. The significant signal represents what is common in all the radar sections, including the break through effect. By subtracting it the anomalies are shown clearly. Usually the average, the median or the minimum value between all the traces are used.

One of the most difficult application for sonic and EM tests concerns the detection of leaves and of their characteristics, (connections, voids, etc...), in a multiple leaf wall or pier. Sonic or radar measurement by transparency or echo are seldom successful due to the fact that: (i) the leaves are parallel to the surface where the antenna is applied, (Figure 5.17), (ii) the rubble and mortars used for the internal leaf are usually made with the same material as the external ones, (Binda 1997b).
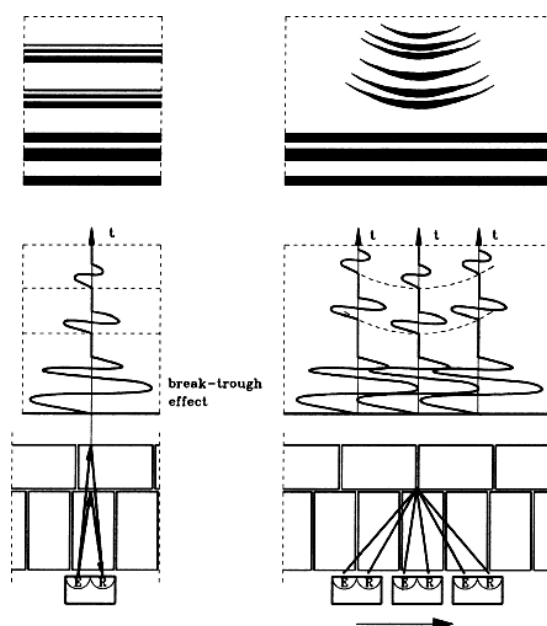
Figure 5.17 Typical features in radar sections due to the leaf and to the joint reflections, (Binda 2000).

For several years the highest frequency for impulsive GPR antennas has been around 1.5GHz. In the last years, GPR suppliers have been marketing new antennas with frequencies higher than 2.0GHz. This advancement is going to favour the analysis of shallow defects like leaf detachments and to extend the GPR potential in studying ultra-shallow details such as estimating the plaster thickness or producing 3D images of the masonry texture hidden by the plaster; this masonry is made of hollow bricks, (Figure 5.18).

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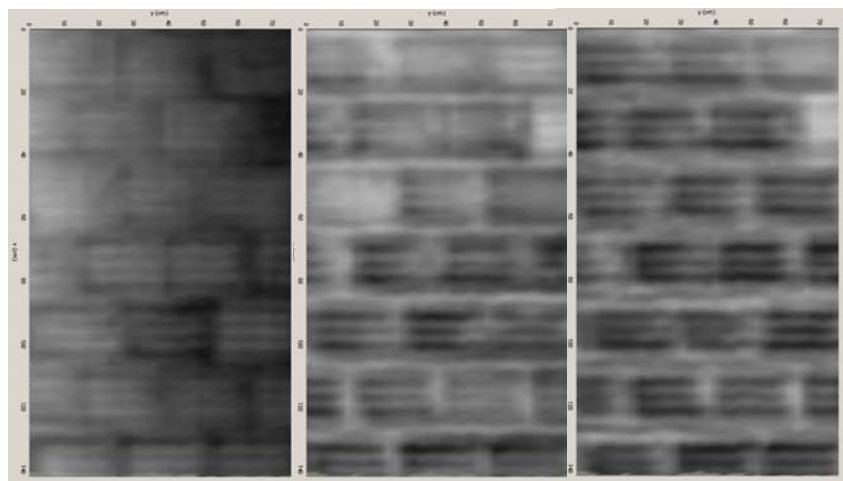
Figure 5.18 Masonry texture behind the plaster (2.0GHz data). Plaster thickness is 1.5cm. Images are shown at 0.61ns, 0.75ns and 0.84ns. The resolution is so high that the image also shows the details of the horizontal holes in the perforated clay bricks, (Binda 2008b).

A very interesting application of radar and 3D data processing was carried out at the Spanish Fortress in L'Aquila which allowed detecting the depth of each stone in the external leaf of the section of the porch pillars, (Figure 5.19), (Binda 2010c).

(a)   (b)   (c)
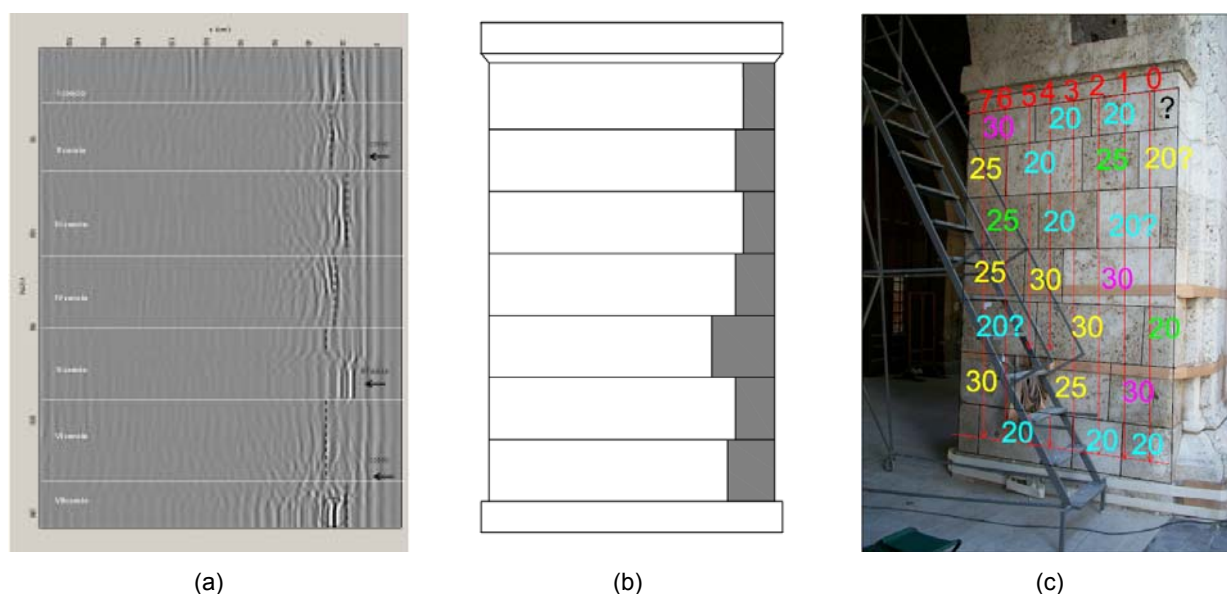
Figure 5.19 Radargram profiles and elaboration by 3D software (1.0GHz antenna): a) Vertical section profiles, b) depth of the external leaf, c) depth of the stones in cm, (Binda 2010c).

5.1.5 Radar and sonic tomography

Among the ND applications the tomographic technique is quite attractive for the high resolution that can be obtained, (Schuller 1995), (Valle 1997). Tomography, developed in medicine and in several other fields, seems to be a valuable tool to give two or and three dimensional representation of the physical characteristics of a solid. Tomography, from Greek "tomos" (slice), reproduces the internal structure of an object from measurements collected on its external surface. Tomographic imagining is a computational technique which utilises an iterative method for processing a large quantity of data. Standard pulse velocity data or radar data could be used to reconstruct a velocity distribution within a solid material, thus providing an "image" of the masonry interior, (Valle 1998).

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Tomography's principle is the Fourier Slice theorem, (Kak 1994), that shows how a complete slice of an object can be extracted from a proper set of measurements. The testing technique gives a map of the velocity distribution on a plane section of the structure under investigation. The method consists of obtaining numerically the time taken by a wave along several directions, which uniformly cover the section under investigation. The computation is made by using the inversion process which, starting from the time of signal propagation, reconstructs the field velocity. The section of the masonry is marked by a mesh grid whose dimension is related to the distance between two subsequent transmission or receiving points. The calculation is carried out in the case of sonic tests, under the hypothesis that (in a non-uniform velocity field) sonic impulses did not propagate in a straight line but following a curved line cause by refraction.

Due to the cost of a tomographic survey, (acquisition time and processing complexity), a good a understanding of which results can be achieved and how is necessary. In fact, the accuracy of tomography depends on many parameters: the source (sonic or electromagnetic), the number and the position of measurements, the equipment settings, the reconstruction algorithms, (Binda 2003d).

It is essential to stress that the resolution capabilities of tomography can be evaluated only taking in account the measurements locations (i.e. the angular distribution of the observations and their spatial sampling) by means of the above theorem. Furthermore the physical limits related to the wavelength should be considered. In fact sonic and EM sources produce diffraction phenomena that limits the resolution. Thus, when the measurements are properly carried out, (i.e. the angular coverage and the spatial sampling honour the Fourier Slice Theorem), the resolution has a physical limit strictly related to the wavelength involved in the survey. On the contrary, when the measurements do not satisfy the above conditions because of the environment or because of the structure geometry, (i.e. in a wall survey), the resolution limits could even be more unfavorable. All these concepts are developed in, (**Errore. L'origine riferimento non è stata trovata.**). The result of the tomographic inversion is a map of a property of the materials. In case of travel time tomography (TT) the measured quantity is the traveltime of the signal and the map is the distribution of the propagation velocity within the object. In case of amplitude tomography (AT) the measured quantity is the amplitude of the signal and the map is related to the distribution of the absorption coefficient.

A special care should be devoted to the: (i) choice of location of measurements, (ii) accuracy in their topological location, (iii) choice of optimal transducers/antennas (operating frequency), (iv) availability of auxiliary tools for a faster and easier acquisition procedure, (v) settings of the acquisition electronic equipment (time range, filters, gains, time sampling, number of bits for A/D conversion). All of those items must contribute to obtain a reasonable number of good sonic/radar signals: the object section will be uniformly illuminated from any direction, (provided that it is possible), and each single trace will contain clear information of its crossed path; hence the signals will be clean with low noise, not clipped and long enough. In such a way it will be easier to extract from the signal all the needed characteristics for inversion.

The point (iii) will be accomplished by a spectral analysis of the received signal; in fact, the choice of an operating frequency as higher as possible suffers the limitation of absorption that is proportional to the frequency and to the object size. Hence the trade-off between resolution, (that requires high frequency), and depth of penetration is a critical aspect in the design of a tomographic survey.

The determination of the absolute time scale is an important item. Usually the zero time does not correspond to the first sample of a trace. It can be obtained in different ways: by coupling the antennas and measuring the cable and the electronic delays, or acquiring a sequential set of traces with antennas located at different distances. In this second case the regression of arrival times versus distance allows to compute the zero time.

The specific characteristics of commercial radar equipment allow using some different tools and procedures to make the radar tomography faster and easier than the sonic one. TT and AT do not require huge pre-processing of the signals. The most critical step in TT is the first arrival picking. In addition AT, (only for GPR), needs the knowledge of the antenna directivity pattern for a correct interpretation of amplitudes.

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Both sonic and radar systems are suitable for travel-time tomography (TT). Usually, materials present complementary behaviors with respect to sonic and EM velocity, (i.e. slow sonic materials are generally fast materials for EM waves and vice-versa). This indicates that one method may be more appropriate than the other depending on the material nature. In some cases, where it is not possible to apply the radar method, (large presence of metals or water), the sonic can be the only solution; vice-versa when the sonic fails fully, (large presence of voids or chaotic in-homogeneities), the radar can detect the main elements of the object section.

In sonic tomography the elastic parameters are expected to be more correlated with the sonic velocity rather than with EM velocity. On the other hand, radar systems, provided the antennas are calibrated, are much more indicated than sonic for the application of other powerful reconstruction techniques such as amplitude tomography, migration and diffraction tomography. Both the sonic and the radar systems suffer the difficulty of the source signature extraction.

Finally, cost considerations are also important and with respect to these it should be pointed out that the GPR equipment is much more expensive but the acquisition times for radar data may be an order of magnitude below the correspondent sonic acquisition times.

In Figure 5.20 an application of sonic tomography representing the distribution of sonic velocities within the section (a) of two church pillars (b and c) is presented as an example.

(a)   (b)   (c)
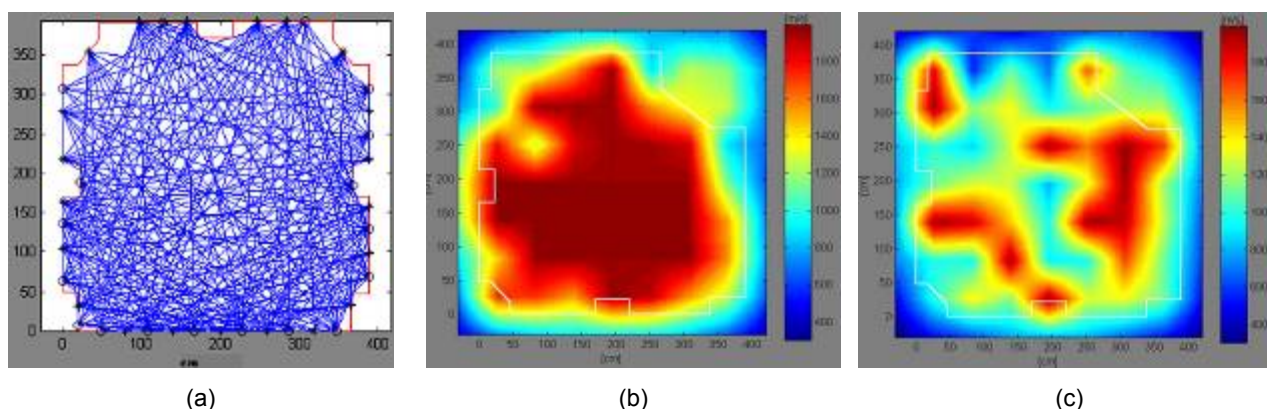
Figure 5.20 Sonic tomography of the horizontal section of two pillars (church of St. Nicolò L'Arena in Catania), (Binda 2003d). (a) Testing points and directions. (b) Pillar 1 at 5.8m height. (c) Pillar 2 at 5.8m height.

5.1.6 Pendulum Hammer Testing

The extraction of large material samples for the purposes of determining mechanical properties is in the case of Cultural Heritage buildings often not permitted nor a preferred solution.

Superficial and non destructive compressive strength determination by means of a pendulum hammer has been found to be suitable for earthen buildings, (Dierks 2000), (Ziegert 2003). The pendulum hammer is a rebound hammer which measures surface hardness, and the particular pendulum hammer used for earthen buildings is a Schmidt type PT, which is specified as appropriate for low strength materials.

The rebound hammer given by the pendulum readings is calibrated for each particular material. Despite the wide scatter of results, by using the average of a large number of readings, one can achieve a good approximate value non-destructively. This was found to be the case in three earthen construction projects adobe (Longfoot 2006), rammed earth (Dierks 2000) and cob (Ziegert 2003), but the technique is not used extensively. The validity of using the Schmidt hammer for adobe is arguable, since the hammer would only assess the compressive strength of the masonry units, not of the homogeneous properties of masonry.

This technique actually determines superficial hardness, and within earth building was initially used successfully for determining the strength of rammed earth in the frame of the Chapel of Reconciliation project in Berlin, (Dierks 2000). Afterwards, it was used for cob, (Ziegert 2003). This

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technique, especially in the case of structures with damaged surfaces, has its limitations. Ziegert (2003) minimises the typical scatter by specifying a minimum of 10 readings in different places.

Figure 5.21 Schmidt Hammer to measure Cob hardness, (Source: ZRS).

By measuring the rebound with the pendulum hammer on samples with differing strength, followed by destructive material testing of the same samples, a calibration equation can be developed to correlate readings with the hammer to compressive strength of earth. Once the compressive strength is known, if the relationships between compressive strength and bending strength, modulus of elasticity or shear strength are also known, a number of mechanical properties can be estimated.

5.1.7 Moisture Assessment

The moisture content of earth structural elements often influences greatly their mechanical properties. Some materials are more influenced in their properties, (e.g. earthen materials), others less, (e.g. dense stone, concrete). When considering moisture in structural elements one has to differentiate between fairly stable equilibrium content in the materials and between a possible additional moisture content by water ingress from the outside. The latter one may fluctuate in time and change the moisture content of a structural element periodically or randomly. When assessing the moisture content in historical buildings it is therefore necessary to know also something about the moisture distribution. For example many historical structures have problems with uprising damp due to moisture ingress via the footings from the ground or by surface water due to a lack of sufficient drainage. In relation of moisture assessment it is necessary to identify moisture distribution vertically and in the cross section of the masonry.

Methods for moisture measurement consist of destructive, (usually one time), methods and low intrusive or non-destructive, (often continuous), methods. Destructive methods require samples, (mostly in powder form), from the actual structural element and determine the moisture content directly. Low invasive or non-destructive methods can either only give relative or indirect moisture readings the latter requiring a careful calibration. The simplest destructive method consists in the gravimetric technique by extracting a sample from a building, weighing it immediately after extraction, dry it and then weigh it again. The difference is the absolute moisture content in g; dividing it by the dry mass and multiplying it with 100 gives the moisture content in percent.

$$u_m = (m_w - m_d)/m_d \cdot 100 \ \%$$ Eq. (5.3)

(u_m moisture content by mass, m_w mass of moist material, m_d mass of dry material)

If the apparent density of the material is known, the moisture content can also be expressed in vol-%:

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$$u_v = u_m \cdot \rho_m/\rho_w \qquad \text{Eq. (5.4)}$$

(u_v moisture content by volume, ρ_m apparent density of material in kg/m^3, ρ_w density of water, $\approx$ 1000 kg/m^3)

Usually drying of the samples takes place in an oven at 105.0°C until mass balance. However, materials containing clay or gypsum need to be dried as low as 40.0°C until mass balance since at higher temperatures not only physically bound water may evaporate but also physiochemical bound water, (e.g. in clays) or crystal water, (gypsum). Another direct method is the CM-technique and is based on the reaction of calcium carbide with water. One of the two reaction products is acetylene (C_2H_2) and the amount of acetylene formed is directly proportional to the water content of a sample. To determine the water content of a sample a CM device is used were the reaction occurs in a pressure vessel and the moisture content is directly recorded by a calibrated manometer. The disadvantage of this method is the low accuracy, in particular if the original water content of the sample is low.

The extraction of samples can be carried out with different methods, either by chisel and hammer, cutting devices, drill core or by drill powder extraction. The first method causes the least impact on the moisture content of a sample the latter ones changes it to different degrees due to heating of the samples during extraction.

Low intrusive or non-destructive techniques are usually applied also for monitoring purposes. A relative method consists in the determination of the relative humidity on the substrate surface or in a borehole within a wall. For this many commercial instruments are readily available. However, results are not very accurate and do not give direct results. Other non destructive techniques consist in *multi ring electrodes* (MRE) and *time domain reflectometry* (TDR). Both techniques allow a continuous monitoring of the moisture content of a building, (Schießl & Breit, 1995; Plagge, 2002). By placing multiple sensors the moisture distribution over time can be monitored. However, both methods are influenced by the ion concentration of the water and need to be calibrated for every type of material, (Zakri & Laurent, 1998).

Thermovision, (Binda 2010b), can be successfully used in the moisture mapping as well as radar systems, (Binda 1998a).

5.2 SLIGHTLY DESTRUCTIVE TECHNIQUES

5.2.1 Single flat jack

The method was originally applied to determine the on-site stress level of the masonry and it has been extended to the detection of its deformability characteristics. The first applications of this technique on some historical monuments, clearly showed its great potential. It appeared to be the only way to achieve reliable information on the main mechanical characteristics of a masonry structure, (deformability, strength, state of stress). The test is carried out by introducing a thin flat-jack into the mortar layer. The test is only slightly destructive. After the test is completed, the flat-jack can easily be removed and the mortar layer restored to its original condition, (ASTM 1991b).

The determination of the state of stress is based on the stress relaxation caused by a cut perpendicular to the wall surface; the stress release is determined by a partial closing of the cutting, i.e. the distance after the cutting is lower than before. A thin flat-jack is placed inside the cut and the pressure is gradually increased to obtain the distance measured before the cut. The displacement caused by the slot and the ones subsequently induced by the flat-jack are measured by a removable extensometer before, after the slot and during the tests. P_f corresponds to the pressure of the hydraulic system driving the displacement equal to those read before the slot is executed. The equilibrium relationship is the fundamental requirement for all the applications where the flat-jack is currently used:

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$$S_f = K_j\, K_a\, P_f$$ Eq. (5.5)

S_f = calculated stress value

K_j = jack calibration constant (<1)

K_a = jack/slot area constant (<1)

P_f = flat-jack pressure

In brick masonry, the cut can be easily made in the horizontal joints. For this type of masonry a rectangular flat-jack is used. The cut can also be made by a steel disk, with a diamond cutting edge. The flat-jack has the same shape of the cut. In fact the use of flat-jacks for stone masonries made with irregular stones is not so easy, due to the difficulty of finding regular joints; therefore the cut for the insertion of the jack is done directly in the stone courses. It must be pointed out that the flat-jack test in the case of multiple-leaf walls gives results concerning only the outer leaves.

The reliable determination of the equilibrium pressure is the fundamental requirement for the test, regardless the details of the type of application, (Binda 1999c). Conflicting information, due to the effect of the concentration of stresses and/or inelastic deformations or to the detection of very low stresses, (e.g. one or two floor building), usually require significant amount of subjective judgement, which may compromise the reliability of the entire procedure, (Ronca 1997).

5.2.2 Double flat jack

The test described can also be used to determine the deformability characteristics of a masonry. A second cutting is made, parallel to the first one and a second jack is inserted, at a distance of about 40.0 to 50.0cm from the other. The two jacks delimit a masonry sample of appreciable size to which a uni-axial compression stress can be applied. Measurement bases for removable strain-gauge or LVDTs on the sample face provide information on vertical and lateral displacements. In this way a compression tests is carried out on an undisturbed sample of large area. Several loading could be performed at increasing stress levels in order to determine the deformability modulus of the masonry in its loading and unloading phases, (Binda 1999c, 2007a).

 Figure 5.22 shows the application of the double flat-jack test in the case of brick masonry with thin and thick joints and of an irregular stone masonry. In the third example the slots have to be made with a special eccentric saw within the stones, being the mortar joint very irregular and weak. The flat-jack is than a semicircular one.
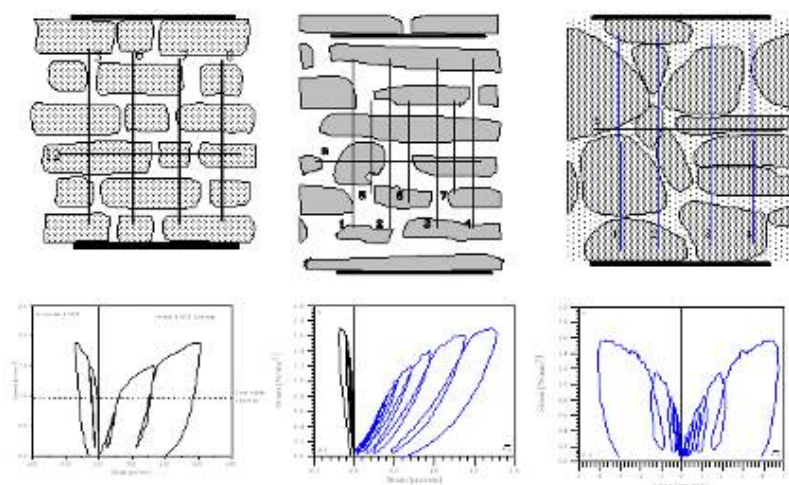

Figure 5.22 Double Flat-Jack test on Monza Tower, S Vitale (Ravenna) and Castelletto (Toscana), (Binda 2000).

Difficulties or impossibility in applying this test can also be found in the case of low rise buildings, (one or two story high), due to the lack of stress contrast in the masonry caused by the low

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stresses found in it. In these cases the situation can be known after carrying out a single flat-jack test and the double jack-test can be either avoided or carried out up to a low state of stress just in order to have information on the elastic parameters.

Remarks can be useful concerning the calculation of the modulus of elasticity from the test results.

Even if ASTM, (ASTM 1991c), proposes a methodology in the elastic modulus computation, conceptual uncertainties concern the detection of the linear behavior of the masonry. In fact, due to the masonry peculiar behavior, the elastic phase is often difficult to estimate. In many cases, locking in the initial phase occurs due to large deformation of the masonry in the first steps of the compression test. Furthermore, since it is impossible on-site to reach the ultimate state of the masonry, changes in the curve slope can also be interpreted with great difficulty. On the base of experience, the definition of the elastic modulus can be made with different types of computation, but in many cases it cannot be univocally defined. Thus it can create subjective interpretation according to the operator expertise bringing to possible different values.

According to ASTM proposal the value of E can be calculated as:

$$E_t = \frac{\delta\sigma_{mi}}{\delta\varepsilon_{mi}} \quad \text{tangent modulus} \qquad\qquad \text{Eq. (5.6)}$$

where $\delta\sigma_{mi}$ and $\delta\varepsilon_{mi}$ are respectively the increment of σ and ε at each step of loading.

$$E_{si} = \frac{\sigma_{mi}}{\varepsilon_{mi}} \quad \text{secant modulus} \qquad\qquad \text{Eq. (5.7)}$$

where σ_{mi} and ε_{mi} are respectively the value of stress and strain reached at step *i*.

While the values given by Eq. (5.6) follow the variation of E along the envelope of the loading curve, it is more difficult to calculate E with Eq. (5.7) particularly when a locking phase is present. Frequently the elastic modulus is calculated as secant modulus in the linear part of the σ-ε diagram. This way the choice of the secant modulus depends much more on the operator decision. Another possibility can be to calculate the secant modulus during the unloading phase, which represents the elastic response of the masonry during unloading. In Figure 5.23a and b the two ways of calculating the modulus are represented. Further elaborations are ongoing in order to better understand the differences. The possibility of a quantitative estimation of the physical-mechanic property of masonries, by the use of sonic tests, were proposed within a research carried out in 1993 in laboratory and on-site tests and on brick masonry of the Veneto Region, (Binda 1993).

Sonic tests in transparency have been carried out on several buildings in order to obtain an average sonic velocity associated to every studied typology. By single and double flat-jack tests the state of stress and the elastic modulus of each wall were also evaluated.

(a)   (b)

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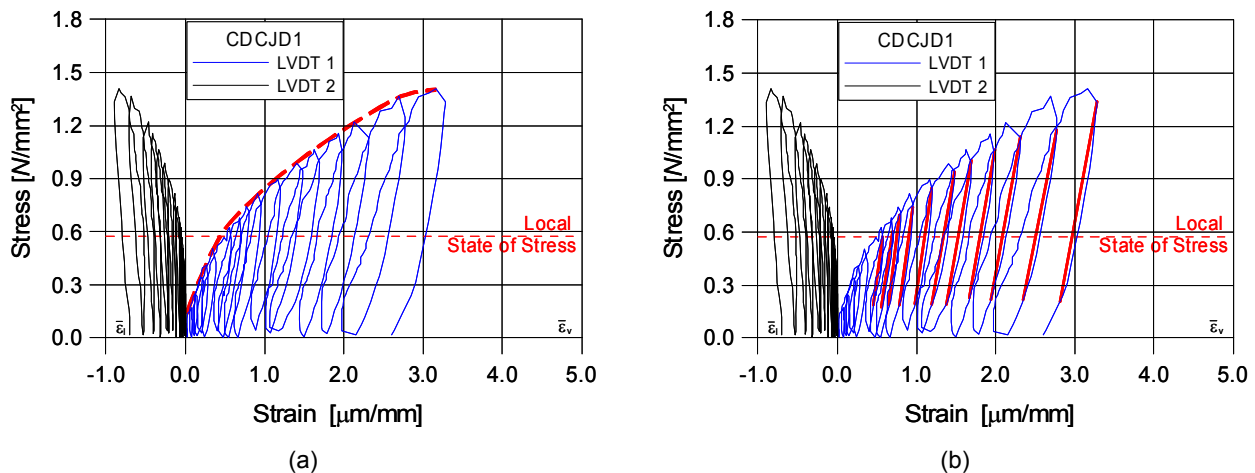
Figure 5.23 Calculation method of: (a) the tangent E modulus according to Eq. (6) and (b) of the unloading secant E modulus. (Binda 2007a).

Figure 5.24a and b shows similar correlation between elastic modulus and the sonic pulse velocity for (a) stonework (regular and irregular) and (b) brickwork. It can be clearly remarked that the stonework gives much higher scattering in the results.

(a) (b)
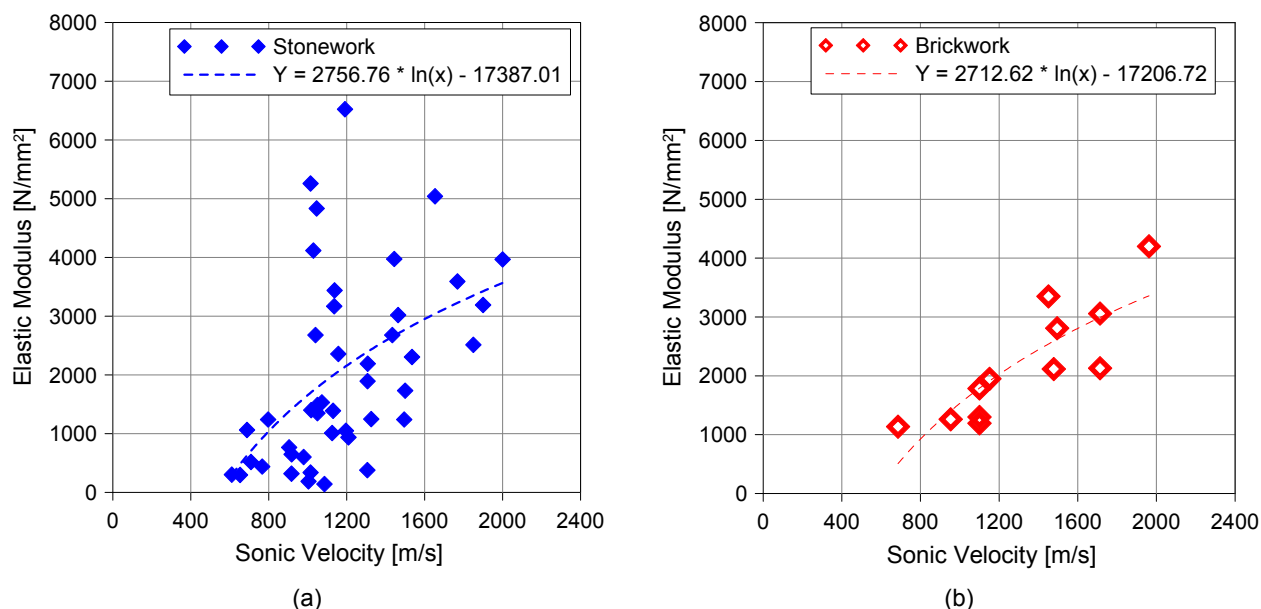
Figure 5.24 Comparison between elastic modulus obtained from the double flat-jack tests and values of pulse sonic velocity measured in transparency in the same area, in case of (a) stonework and (b) brickwork. (Binda 2007a).

5.2.3 Hardness, drilling energy and pull out tests

Other slightly destructive tests can be used to give more information to on-site about masonry and masonry components. They can be considered as surface or small penetration techniques, which can be used for a preliminary investigation. Some of them can be remembered here:

i.   the Schmidt hammer rebound test to detect the quality of mortar joints, has some limits in the present equipment which was set up to be used on cement mortar and can have too high energy for a lime mortar;

ii.   the penetration tests proposed in different ways, like probes, drillers, etc... correlate the depth of penetration to the material mechanical properties. Unfortunately a correlation is impossible to the real strength of ancient mortars; so the calibration of these tests is very difficult. Furthermore the depth of penetration is low, so only the repointing mortars are usually detected;

iii. the pull-out tests can only be used on bricks and stones, very rarely on mortar joints, unless they are not very thick.

Other surface tests have been proposed so far; all of them can be useful to have an overall rough idea of the masonry condition on the surface and they can be meaningful for a preliminary survey of the structure, but they only give the possibility of qualitative interpretation of masonry condition. All these tests can be really useful for the quality control of new masonry.

5.2.4 Core drilling, boroscopy and videoboroscopy

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To understand the morphology of a masonry wall it is important a direct inspection. Sometimes it could be performed by removing few bricks or stones and surveying photographically and drawing the section of the wall.

In some cases it is possible to core boreholes in the most representative points of the walls. Coring should be done with a rotary driller using a diamond cutting edge. This operation is rather simple but has limits. The drilled core is usually very decohesioned so it is almost impossible to detect the quality of the original materials. Inside the boreholes additional investigations can be made by the use of borescopy, (Figure 5.25a), (Binda 2009d). A small camera may be inserted into the borehole allowing a detailed study of its surface and try a reconstruction of the wall section, (Figure 5.25b). The results of this study may be recorded in a video-cassette for further analysis. The information obtained includes the measurements of large cavities and a general view of the materials. Nevertheless the interpretation of the photograms is a very difficult operation, sometimes hopeless; it should be remembered that boroscopy can only give a stratigraphy of the section, (Figure 5.26).
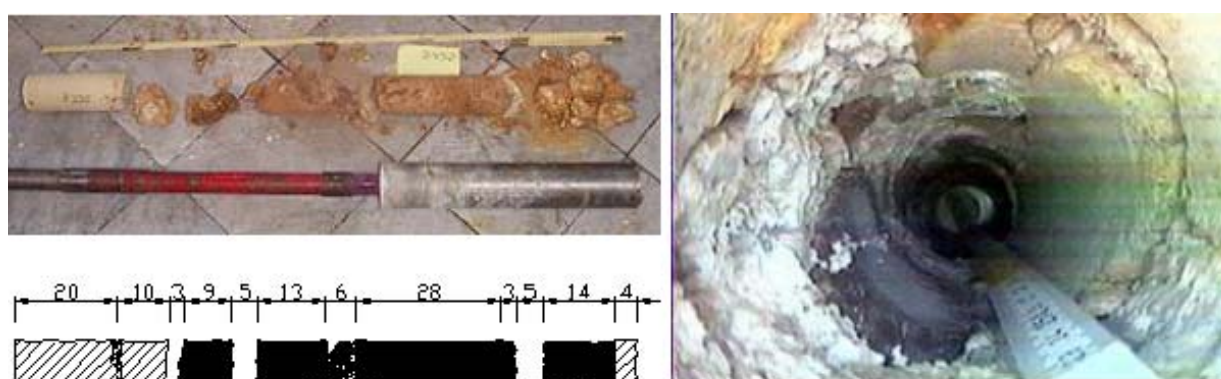
Figure 5.25 (a) Coring of a wall and reconstruction of the section. (b) The inspection by videoboroscopy. (Binda 2009d).

5.3   MASONRY QUALITY DEFINITION

The influence of the masonry characteristics on the physical and structural behavior of load-bearing walls is fundamental. Masonry, especially in historic buildings is often a complex composite material not only due to the variation in origin and properties of its components (brick, stone, mortar), but also to the multiple technique of construction of the wall. In the case of old stone masonry the walls are frequently built with the multiple leaf system, the stones are irregularly cut and the irregularities of the courses are filled with thick mortar joints, while the leaves can be badly connected along the depth of the section. The behavior of these walls is influencing the safety of the building not only under the vertical actions, but especially under the horizontal actions during seismic events. In the last case the effect of this behavior can bring to the separation of the leaves and the overturning of the wall under out-of-plane actions, (Figure 5.26).

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Figure 5.26 Separation of masonry leaves and collapse under out-of-plane actions, (Binda 2006a, b).

Therefore both for repair and prevention purposes, a good knowledge of the masonry characteristics is needed.

The possibility of gaining this knowledge is given by tests which can be carried out on-site and in laboratory; the tests on-site should be non destructive or slightly destructive, while the laboratory tests are carried out on specimens sampled directly from the walls; sampling has to respect the existing wall as much as possible, therefore the quantity of the sampled material has to be minimal. This last recommendation obviously excludes sampling of masonry prisms to be tested in laboratory.

Nevertheless the quality of the masonry wall can be detected first of all by knowing its morphology, (prospect and section), the physical, chemical and mechanical properties of its components and the properties of the masonry as a composite material. It is easy to describe how to tests the material in laboratory, but the on-site testing has to be based on non destructive or minor destructive techniques, (NDTs, MDTs). Among the proposed techniques the most effective are without any doubt the sonic and flat-jack tests.

5.3.1 Methodology of investigation

It is well-known that the masonry mechanical characteristics cannot be deduced by the ones of the components, nor by using existing standards for new masonry, or through laboratory tests on materials sampled from the walls. So only on-site testing on masonry are possible and they have to be non destructive. Up to now the only test which can characterize the state of stress and the stress-strain behavior of a masonry is the flat-jack test (single and double), (ASTM 1991b, c), (Figure 5.8), which is a carried out locally, helped by sonic pulse velocity tests to determine the density and homogeneity of the material, (Binda 1999, 2001).

Of course perhaps more reliable, but also much more destructive tests are available as shear strength diagonal tests and compression-shear tests carried out on-site, (Binda 2005b).

Following the proposal made by the 2003 Italian Seismic Code produced by the Civil Protection Department, (O.P.C.M. 2003), (PCM 2005), to adopt three different levels of knowledge, DIS, POLIMI researchers have tried to propose a methodology after the long experience of on-site testing they have developed in 1996 in Tuscany, Umbria after the 1997 earthquake, (Cardani 2004), (Binda 2005c), Liguria, Lombardia after the 2004 earthquake, (Binda 2007c) and Abruzzo after the 2009 earthquake, (Binda 2009b).

The Italian Code (O.P.C.M. 2003), (PCM 2005), asks for information on: (i) the evolution of the building through archive documents, (ii) the building geometry and the details on connections between walls and walls and floors and roof, (iii) the crack pattern and damage survey. Furthermore a knowledge is required on the *quality* of the masonry. What the request means about

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"quality" can be easily clarified. The knowledge should be extended to: (i) the masonry morphology, (ii) the technique of construction (single, multi-leaf), (iii) the component properties, (iv) the mechanical properties of the masonry under horizontal and vertical actions.

The requested knowledge can be reached first of all by surveying the masonry not only superficially through the prospect, but by "looking inside" in order to detect how the masonry section is made. The section geometry is a parameter for the structural analysis and it is also important for the choice of the type of intervention when necessary, (Binda 2003a), (Binda 2006c), (Binda 2009a, d).

For the choice of an appropriate analytical model the constitutive laws of the materials are needed; therefore the highest number of mechanical parameters is known the most reliable can be the chosen model, (Binda 2006c).

Even if very few parameters can be driven from on-site and laboratory tests, nevertheless the experience has shown that two types of tests can be useful on-site: the sonic pulse velocity test and the single and double flat-jack test, (Binda 2008a), (Binda 2009b,g). The first one is a qualitative procedure which can be useful, when carried out by *transparency,* to find though the distribution of the calculated velocities, (Figure 5.27), the differences in density across the wall. When low velocities are detected, (area A in Figure 5.27 b), the masonry might have voids and defect inside, when velocity high peaks are found there might be a connection between the masonry leaves along the section, (area B in Figure 5.27 b). Of course the results do not give the morphology of the masonry section, which has to be found in other ways. The test can be useful to locate the position of the flat-jack test. The single flat-jack test allows to calculate the state of stress, (Figure 5.27 a), in a compressed masonry (by dead loads) and the double flat-jack test allows to find the stress-strain behavior of the masonry, (Figure 5.27 a). What is needed more in order to qualify the masonry are the section morphology and the properties of the components which can be found by small dismantling of the section and by sampling from the inside mortar, brick and stones. If all the mentioned information can be referred to the same area of the masonry, then the quality of it can be completely studied.

(a)

(b)
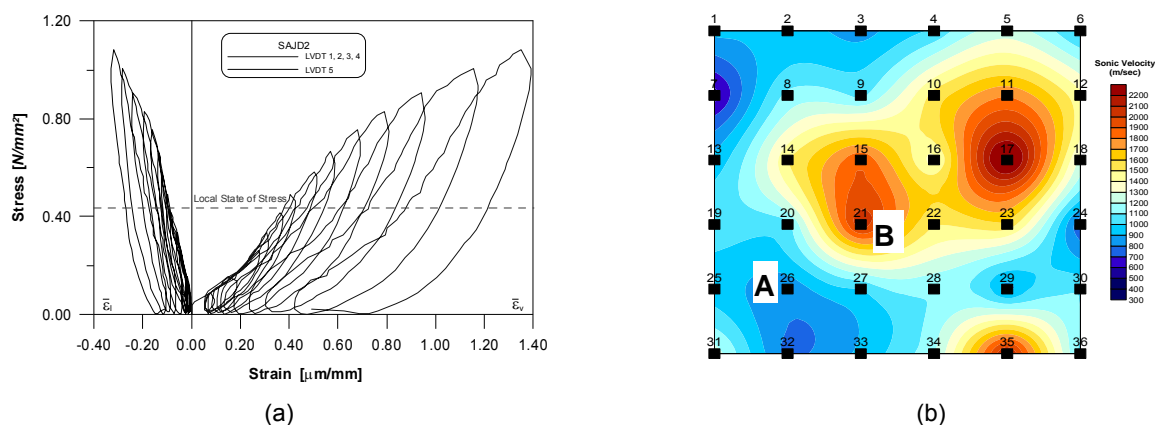
Figure 5.27 (a) Result of flat jack-tests, (b) Sonic velocity distribution. (Binda 2009b).

It is very important for the chosen area to be representative of the masonry under investigation; the problem on how to find a representative area arises in the case of ancient or old buildings especially in seismic areas which where several times modified or partially reconstructed in the past and in which different types of masonries can be found even in the same wall.

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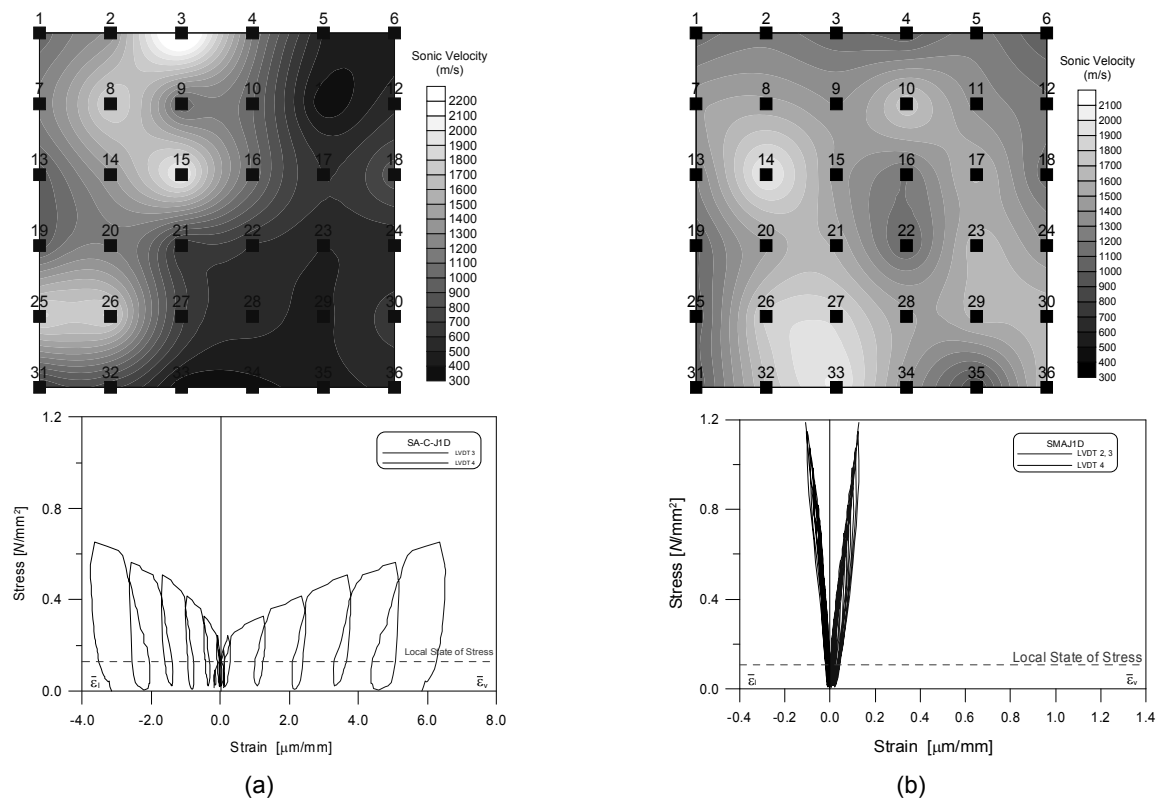
Figure 5.28 Results of sonic and flat-jack tests on the masonry of two churches in Lombardia, (Binda 2009g).

The proposed investigation is carried out according to the following steps: (i) choice of the strategic points on all the types of load bearing masonries in the building, (e.g. choice of a point for each masonry texture observed through the prospect, at the lowest most loaded part of the wall), (ii) survey of the masonry texture of the chosen area in prospect, (iii) sonic pulse velocity test by transparency on a grid of 1,0mx1,0m including the area chosen for the flat-jack test; the velocity peaks as said above will indicate a higher density of the material, perhaps the presence of a continue stone or a course of stones crossing the whole section, (iv) single flat-jack test to define the state of stress of the masonry in the chosen area, (v) double flat-jack test with the collection of data, appropriate drawing of the stress-strain curve and calculation of Young modulus and Poisson's ratio; on the diagram also the calculated value of the stress, (by single flat-jack), should be reported in order to see the residual load-carrying capacity of the masonry in the elastic state, (Figure 5.27). The sonic velocity distribution should also be represented, (Figure 5.28 a,b), (vi) small dismantling of one or two stones through the section up to ¾ of its thickness, possibly made in correspondence of the sonic velocity peak, (Figure 5.29), (vii) graphic representation of the prospect and section of the wall, (viii) sampling of a stone and of mortar from the internal part of the masonry in order to be sure that they are the original ones, (ix) chemical, petrographic analyses on mortars, physical and mechanical tests on mortars and stones in order to define their composition and origin in view of a future intervention, (x) repair of the small damage caused to the masonry by sampling, (Binda 2009b, e, f).

It is possible to see from Figure 5.28 that the highest sonic velocities, (Figure 5.28b), also correspond the highest elastic modulus, (Figure 5.28b); hence there is a way to define the best quality of the masonry that one which has high sonic velocity and high modulus of elasticity. The description of the masonry quality is completed by the section morphology, (Figure 5.29), and of course also by the results of the laboratory investigation on mortars and stones.

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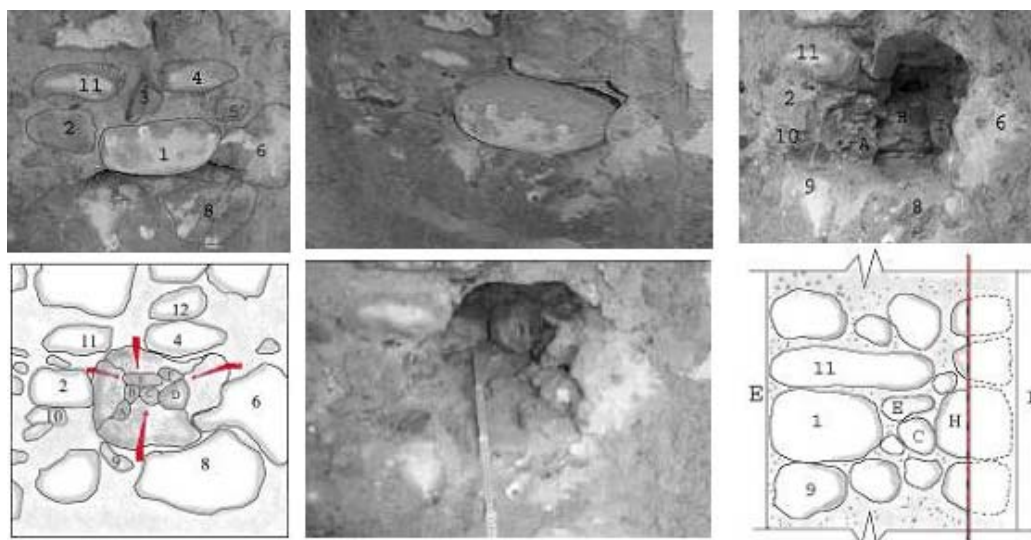
Figure 5.29 Study of the morphology of the masonry section. Drawing of the masonry texture in the section, (Binda 2009b).

In Annex 2 a special template proposed within the ReLuis project in Italy is attached which can be useful to collect data on the masonry quality and can be part of a Data Base, (Binda 2006d, e).

5.4   LABORATORY TESTS

If samples of the materials are needed for destructive tests they must be cored from the walls inflicting the lowest possible damage. The technique of sampling is very important, since samples must be as undamaged as possible in order to be representative of the material on-site. The aims of these tests are the followings: (i) to characterise the material from a chemical, physical and mechanical point of view, (ii) to detect its origin in order to use similar materials for the repair, (iii) to know its composition and content, and (iv) to measure its decay and the durability to aggressive agents from new materials used for restoration. Since it is very difficult to sample prisms representative of the walls, only single components or small assemblages are removed.

5.4.1   Sampling of bricks, stones and mortars

The method of sampling depends on the characteristics of the materials on-site. Some simple principles have to be applied:

1. sampling must be carried out respecting the existing building;
2. the quantity of sampled material must be consistent with the scope and the requirements of the test procedures;
3. if determination of the type and the extent of damage is involved, sampling must be carried out on different portions of the building in order to study all the types of degradation;
4. sampling has to be carried out dry in portions of the building not subjected to the action of rain or by a previous repair, especially when mortar binder and aggregate characteristics are needed;
5. the number of samples should be quite high, in order to represent statistically the situation of the existing masonry.

5.4.2   Tests on mortars

Nowadays there are no standardised tests to define the composition and the chemical-physical and mechanical characteristics of mortars sampled from an existing building. It is often very difficult

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to drill samples having the consistent dimensional tolerance needed to conduct mechanical tests; then the only useful information which can be obtained concern the mortar composition and the state of decay. Chemical and mineralogical-petrographical analyses are useful, (and less expensive than other more sophisticated tests), to determine: the type of binder and of aggregate, the binder/aggregate ratio, the extent of carbonation, the presence of chemical reaction which produced reaction products, (pozzolanic reactions, binder-aggregate reactions, alkali-aggregate reactions), (Binda 1988).

If only a minimum amount of sample material is available thin section microscopy can be in particular valuable. Optical microscopy on thin sections can identify the type of binder, (lime, hydraulic lime, gypsum, cement, earth), the type of aggregate, (carbonate, siliceous, porous, non-porous) and the general binder, aggregate and air void ratio in a fast and efficient way, (Elsen, 2006; Müller, et al, 2008a; Müller & Weise, 2008b). The method is astonishingly accurate and often more suitable than other phase analytical techniques, such as X-ray powder diffraction or infrared spectroscopy, since not only compositional aspects are analyzed but also the mortar texture including air voids and capillary porosity, cracks and reaction structures, (Figure 5.30 and Figure 5.31). Binder/aggregate ratios and the amount of air voids can be determined by quantitative microscopy as described in Larbi & van Hees (2000) and given by the proposed method RILEM COM-C1 (2001).

The grain size and distribution of the aggregates can also be measured by separating binder from aggregates through chemical or thermic treatments, (Baronio 1991). The above mentioned tests permit the determination of the composition of the existing mortars and permit the reproduction of mortars and grouts for repairing the masonry, (Baronio 1999).

However, this method isn't successful when many carbonate aggregate grains are present in a mortar and the procedure is not accurately followed. Too aggressive chemical treatment could dissolve carbonate aggregate and high thermal treatment might also destroy porous or clay containing carbonate grains. In (Baronio 1991) the limits of the procedure, as well as the details of the calibrated methodology are commented. An indication of the sieve curve in a given mortar sample may then be recovered from analyzing the particle size distribution of the aggregates by digital image analysis from thin section photo micrographs, if the grain size distribution is limited.

The mortar porosity can be derived from the apparent density and if the absolute density of the mortar is known, porosity can be directly calculated. The latter one can be determined even from smaller pieces by hydrostatic weighing, (e.g. according to EN 1936). At the same time the total water absorption under atmospheric condition can be measured.
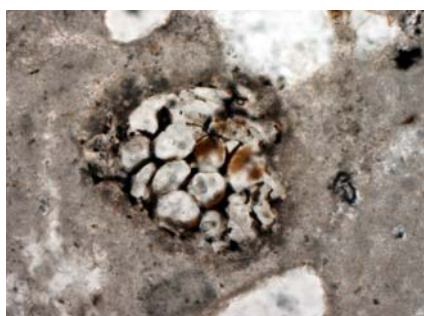
Figure 5.30 Hydraulic component (calcium disilicate) in a historic natural hydraulic lime from a bridge from the 16th century (photo micrograph from thin section, transmittel light, source: BAM).
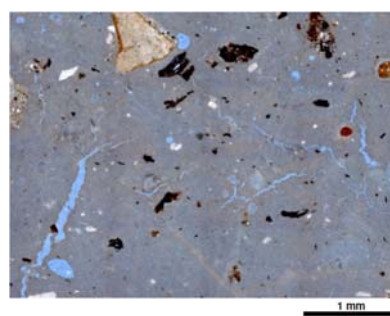
Figure 5.31 Extremely binder rich historic lime mortar from a Arabic citadel from the 9th century (photo micrograph from thin section, transmittel light, source: BAM).

The above mentioned tests permit the determination of the composition of the existing mortars and give information for the reproduction of mortars and grouts for repairing masonry, (Baronio 1991, 1999). If mechanical tests on the original mortars cannot be performed due to the lack of suitable specimens their binder composition in conjunction with their porosity/apparent density can give hints on strength characteristics, (Müller & Kanan, 2005a; Müller, et al., 2005b). So will high lime,

weakly hydraulic and earthen mortars usually yield the lowest strength followed by hydraulic and gypsum mortars and with cement mortars yielding the highest strength.

When considering deterioration processes on masonry the content of salts in the mortar should be determined, (Bläuer Böhm, 1996). This is carried out by eluting a known amount of ground sample with a defined amount of water, thus dissolving the salts. The ions in the eluate can then be determined either by photospectral methods, atom absorption spectrometry, ion chromatography or other chemical analysis methods. Mortars can easily transport salts or can be a source of salts themselves, e.g. in cementitious mortars, (Müller et al., 1997a), or by preparing the mortars with salt laden water, (Müller, et al., 2002).

5.4.3 Tests on damaged and new bricks and stones

The assessment and analysis of the block materials of masonry and its condition concerning non-structural damage is a very complex task. The strategy would be a two step approach:

- an on-site condition assessment in form of a damage mapping in order to identify the extent and type of damage;
- in-depth laboratory analysis for characterising the material properties and the causes of deterioration and damage, (e.g. by salts, frost or hygrothermal action).

Concerning condition on-site assessment of stone masonry many approaches were pursued. A very detailed scheme for the assessment of non-structural damages was developed by Fitzner et al. (1995). This method records not only the type of damage but also incorporates their quantification, (as long as the damage is visible on the surface, Müller et al., 1997b; Müller, et al. 2002). The assessment method is based mostly on visual examination of each stone block and is assisted by taping the stone surface by a hammer in order to detect hollow places of partially detached stone. At the same time different features can be recorded, e.g. the type of damage and the type of stone, (Figure 5.32). The method is nowadays sometimes aided by more sophisticated methods such as ultrasound measurements or passive or active thermography.

(a)

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(b)
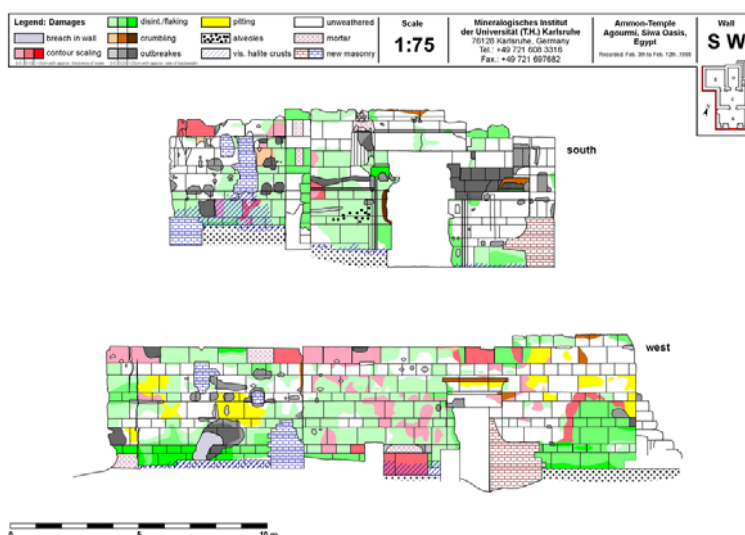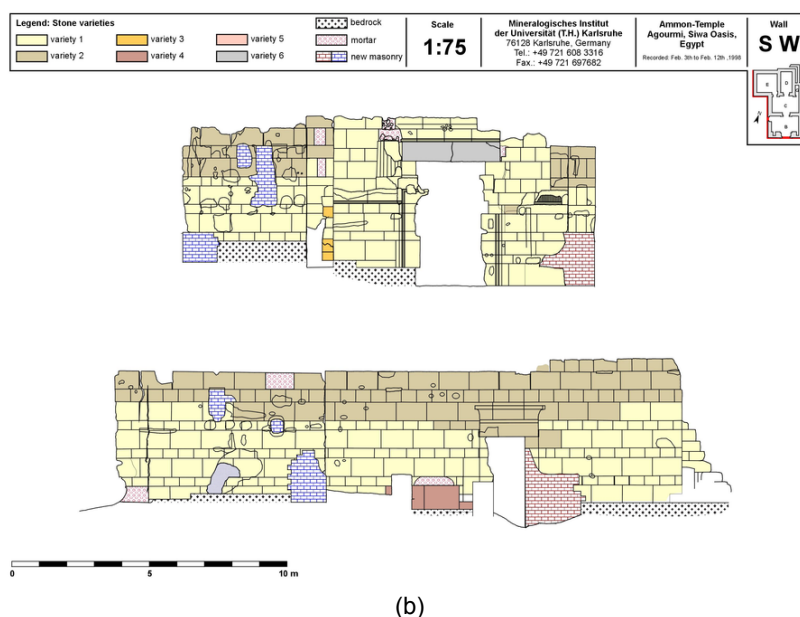
Figure 5.32   Example for Condition on-site assessment of a stone masonry: (a) Distribution of damage types. (b) Distribution of stone varieties. (Müller et al., 2002).

When masonry is damaged by aggressive agents the decay is never uniform; if maintenance is needed and only some bricks or stones or decorations are affected by the damage, the best remedy is frequently the substitution of the most decayed elements. In this case, laboratory tests can give useful information for the choice of the appropriate material for substitution.

When substitution is not possible and/or the decay is very extensive, a surface treatment may be required; again laboratory tests are needed for the right choice of the treatment. The tests have to be carried out on both deteriorated existing bricks or stones, and on undamaged and new ones. The following tests are suggested:

- Mechanical tests: compressive and indirect tensile tests, hardness tests on different points of the brick or stone sections in order to determine the depth of the decay;
- Physical tests: the volumetric mass, the water absorption by total immersion, the water absorption by capillary rise are important characteristics needed to determine the durability of the materials and the effects of surface treatments; the initial rate of suction of bricks and stones and the water retentivity of mortars can be useful when choosing mortars and grouts for repairs; the X-ray diffraction measurements can detect the type of salts found inside or on the surface of a decayed masonry; mercury porosimetry is a useful technique for evaluating durability and surface treatments; thermal and water expansion coefficients must also be measured on new bricks and stones;
- Chemical tests: tests for alkaline sulfate can be conducted on material samples taken at different depth of the masonry in order to detect the presence and quantity of these very aggressive salts;
- Optical and mineralogical analysis: optical observations, (stereomicroscopy, SEM microscopy) define the deterioration, its causes and the presence of salts. Petrographic observations on thin sections determine the pore size and distribution of the material, the size and distribution of the aggregates, the geographical origin of clays and stones, the firing temperature of bricks and the decay and its causes;
- Durability tests: freeze/thaw and salt crystallization tests are needed for new bricks and stones in order to determine their performance under aggressive agents.

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5.4.4 Tests on original earthen materials and earthen repair materials

For the determination of the properties of earthen materials a variety of other strategies has to be applied. Traditionally many testing and analysis methods for earth were deriving from soil mechanics. However, these tests are not always meaningful in the context of describing the properties of earth as a construction material. In order to maintain comparability with other materials in essence the same or similar test can be performed as for stone, brick or mortars.

The apparent density of historical earth can be determined very accurately according to the standard ASTM D 7263-09. The method is based on hydrostatic weighing of earth samples, (regular or irregular in shape), which are coated in order to prevent dissolving in water. However, other as stated in the standard drying of the samples should not be performed at 105.0°C but at 40.0°C in order to prevent irreversible evaporation of water from the earth. The porosity can then be calculated according to EN 1936.

Mechanical tests can be carried analogue to stone or brick if enough sample material is available. The sample size has to be adjusted to the type of earthen materials. Cob or rammed earth with their larger features will require a larger specimen size than a finer grained adobe block material. Also, when performing mechanical tests on earth materials prior conditioning is crucial for good results. All earthen materials will show a dependency of strength with their moisture content, therefore storing at constant relative humidity until mass balance is mandatory before testing. For earthen materials a climate of 23.0°C/50.0% R.H. was found to be most suitable, (Ziegert et al., 2010).

For characterizing the composition and texture of earth a variety of the previously discussed methods can be utilized. The presence of salts can be quantified in the same way as for mortars. For determining the composition of earthen materials mostly phase analytical methods are employed. The x-ray powder diffraction (XRD) method is suitable not only to describe the main constituents but also for the characterization of the binder fraction. For a proper measurement the latter has to be separated from the bulk of the material in order increase the sensibility of the measurement. This is usually done by sieving and centrifugation, (Tribut & Lagaly, 1991). After separation of the clay fraction a textured sample specimen is prepared. This is done by sedimentation of a small volume of the clay suspension on a round glass slide. After evaporation the specimen is being analyzed. In order to detect swelling clays, the sample is stored in an ethylene glycol atmosphere after the first analysis and then measured again. In the presence of swelling clay minerals the according peaks will be shifted. Figure 5.33 gives an example of XRD patterns of earthen materials.

(a)

(b)
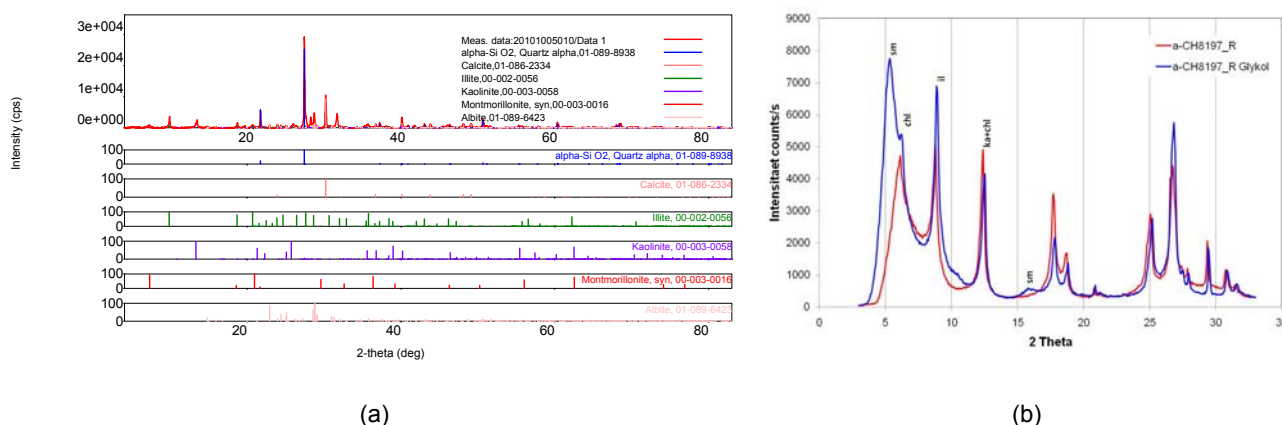
Figure 5.33 Examples for the results of XRD analysis of earthen materials. Left: Bulk analysis of the total earth. Right: Analysis of the clay fraction only (sm = smectite, chl = chlorite, il = illite, ka = kaolinite). Data from BAM.

Sometimes the particle size distribution (PSD) of the original earth is useful to know. This is in particular the case if repair materials need to be applied to existing materials. The particle size

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distribution can be determined according to different standards, (DIN 18123, ISO 11277, ASTM D422 - 63). Usually the test method consists of two parts, first a sieve analysis for particles > 0.1mm and a sedimentation analysis for particles < 0.1mm. However, the definition of the size fraction range is different in the Americas and in Europe:

	Gravel	Sand	Silt	Clay
Europe	> 2.00 mm	2.00 to 0.063 mm	0.063 to 0.002 mm	< 0.002 mm
ASTM	> 4.75 mm	0.075 to 4.75 mm	0.075 to 0.005 mm	< 0.005 mm

Figure 5.34 illustrates an example for the particle size distribution of historical earthen materials. The PSD is usually represented in form of a sum curve, (Figure 5.34). These curves allow an immediate determination of the amount of the respective size fractions.
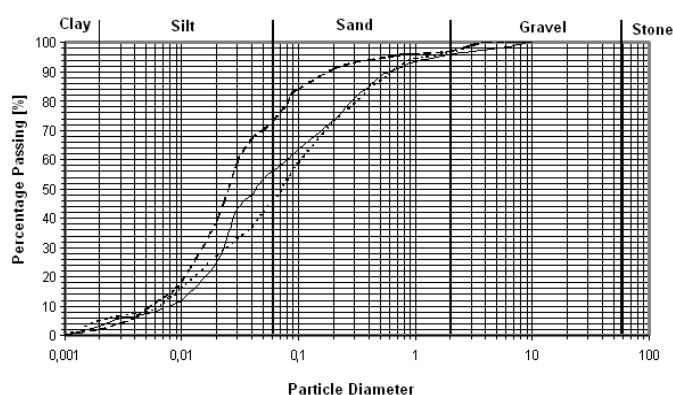
Figure 5.34 Particle size distribution of historic earthen materials, (source: Christof Ziegert).

As for mortars optical microscopy can also be employed for earthen materials. This is in particular meaningful if only a limited amount of sample materials are available. Thin section microscopy gives information on the type of sand and silt sized fraction but not on the type of the clay fraction. The latter one is below the resolution of the optical microscope. However, additional results concerning the earth's texture and additions, (e.g. fibers), can be obtained by the method, (Müller & Hansen, 2001; Figure 5.35). Electron microscopy, e.g. by means of a scanning electron microscope, doesn't usually provide good results since soil clay minerals are in particular intermixed with each other and lacking of any descriptive features.
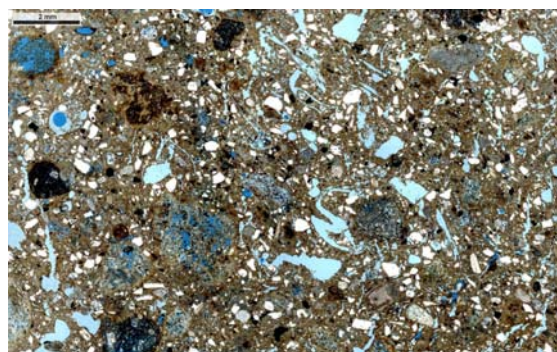
(a)
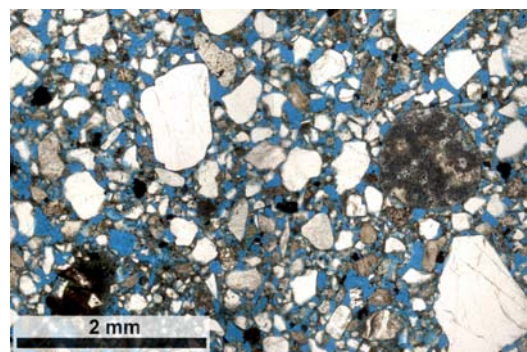
(b)

Figure 5.35 Photo micrographs of the material texture of historic cob (left) and historic adobe (right). The blue colour indicates the porosity, (transmitted light, source Urs Müller).

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For repair materials the cohesion test can be useful in order to evaluate the amount of cohesive binder components and also if a soil needs to be amended by e.g. sand or fibres. The cohesion is tested with specimens moulded in the shape of a bone, (Niemeyer 1944 and 1946). Niemeyer (1946) qualitatively categorises building earth on the basis of a quantitative measure of cohesion [g/cm²], according to a scale from "lean" to "fat" clay, (Table 5.1).

Table 5.1 Classification of construction earth according to cohesion.

Description	Cohesion (g/cm²)
Very lean	50-80
Lean	80-110
Nearly fat	110-200
Fat	200-280
Very fat	280-360
Clay	>360

The type of clay suitable for each type of construction technique is then specified by guidelines, (e.g. Volhard & Röhlen, 2009). For instance, rammed earth can be built with earth from "lean to fat", i.e. cohesion from 80 to 280g/cm² and cob can be built from 50 to 280g/cm².

Further methods for the analysis of earthen materials by simple means, which can be applied on-site and are mostly applicable for repair materials can be found in Houben & Guillaud (1994) and Minke (2000).

5.4.4.1 Tests on extant earthen construction materials for the determination of a suitable repair material

5.4.4.1.1 Granulometry/Grain size distribution

Granulometry after DIN 18123, where sieve analysis is combined with sedimentation analysis, to determine, particle composition according to the following:

Clay particles, dia < 0.002mm

Silt particles, 0.002<dia<0.06mm

Sand particles, 0.06<dia<2.mm

Gravel particles dia>2.0mm, gravel and stone

5.4.4.1.2 Cohesion test

According to the preliminary German standard DIN 18952, sheet 2, and to the current earth construction rules used in Germany, (Dachverband Lehm 2009), cohesion is measured by means of the "Achterling", (i.e. "little 8" test), after Niemeyer, (Niemeyer 1944, 1946), which qualitatively categorises building earth on the basis of a quantitative measure of cohesion [g/cm²], according to a scale from "lean" to "fat" clay.

The type of clay suitable for each type of construction technique is then specified by the standard. For instance, rammed earth can be built with earth from "lean to fat", i.e. cohesion from 80.0 to 280.0 g/cm² and cob can be built from 50.0 to 280.0 g/cm².

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Table 5.2 Classification of construction earth according to cohesion.

Description	Cohesion (g/cm²) from "Achterlinge" test
Very lean	50-80
Lean	80-110
Nearly fat	110-200
Fat	200-280
Very fat	280-360
Clay	>360

5.4.4.1.3 Plasticity

Atterberg Limits for earthen construction are specified by the Lehmbau Regeln, (German "Rules" for building with earth). The applicability of the results is limited.

5.4.4.1.4 X Ray Diffractometry (XRD)

In order to understand which fine grained particles constitute an earth building material, its mineral composition, i.e. its composition in terms of individual constitutive minerals, can be determined by means of x-ray diffractometry.
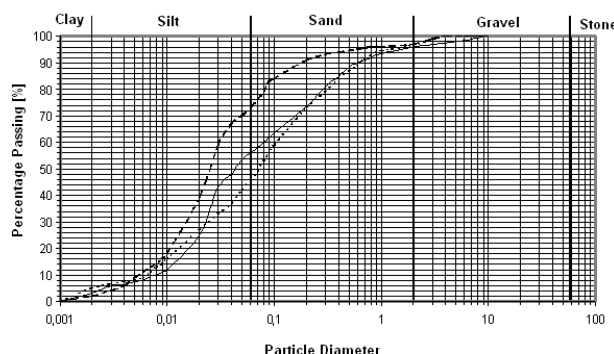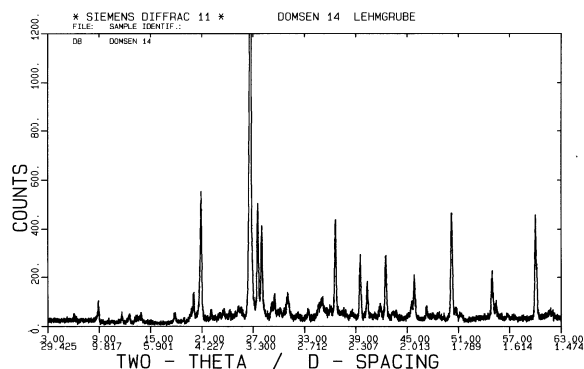
(a)   (b)

Figure 5.36 Grain distribution (left) and X-Ray Diffractometry (right) for different historical cob samples, (Source: Ziegert Seiler Ingenieure, Partner 17).

After granulometry tests have determined the particle distribution of a clay, X-ray powder diffraction can be carried out to determine the mineral composition of its fine-grained particles, which are by then powder samples. Powder x-ray diffractometry is a scientific technique widely used for the characterization of micro-crystalline materials that uses X-ray diffraction on powder samples for the mineralogical-structural characterisation of materials, (Louer 1999). The phenomena by which X-rays are reflected from the atoms in a crystalline solid is called diffraction. The diffracted X-rays generate a pattern that reveals the structural orientation of each atom in a given compound.

XRD finds the geometry or shape of a molecule using X-rays. The X-rays get diffracted by a crystal because the wavelength of X-rays is similar to the inter-atomic spacing in the crystals.

When the X-ray beam encounters the regular three-dimensional arrangements of atoms in a crystal, most of the X-rays will destructively interfere with each-other and cancel each-other out, but in some specific directions the X-ray beams interfere constructively and reinforce one another. It is these reinforced diffracted X-rays that produce the characteristic X-ray diffraction pattern.

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For most applications it is essential that the powder diffraction data be collected appropriately. Therefore, it is of prime importance to spend time optimizing the adjustment of the diffractometer, the quality of the radiation employed and the randomization of the crystallites in the sample. There are several designs for X-ray powder diffractometers, (reflection or transmission modes), each of them having advantages and disadvantages.

Figure 5.36 shows an optics arrangements commonly used with conventional divergent-beam X-ray sources, based on the Bragg-Brentano parafocusing geometry. The beam converges on the receiving slit after diffraction by the sample. The geometry is characterized by two circles, the goniometer circle with a constant radius R and the focusing circle with a radius dependent on y. It uses a flat sample which lies tangentially to the focusing circle. The main advantage of this reflection geometry is that no absorption correction has to be made if an 'infinitely' thick sample is used. Disadvantages are that by using a flat sample, preferred orientation effects are increased and at low angles the illuminated area can become larger than the sample. Preferred orientation effects can be reduced by using side-loaded sample holders.
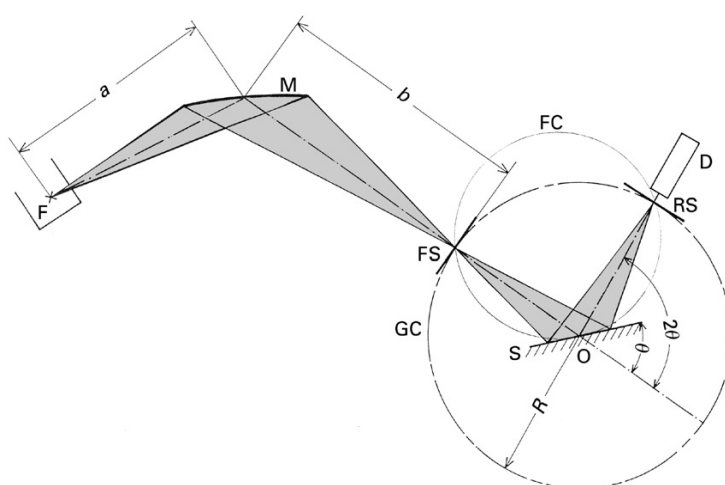
Figure 5.37 Optics of a conventional focusing powder diffractometer with monochromatic X-rays, (Bragg-Brentano geometry with a reflection specimen): F line focus of X-ray tube, M incident-beam monochromator, a short focal distance, b long focal distance, FS focal slit, S flat specimen, GC goniometer circle, O goniometer axis, R goniometer radius, FC focusing circle, Bragg angle, 2 reflection angle, RS receiving slit, D detector, (Louer 1999).

X-RAY Diffractometer at BAM: The Theta-Theta Type X-ray Diffractomator Ultima IV Rigaku Corporation

The Theta-Theta Type X-ray Diffractomator Ultima IV Rigaku Corporation is a High-precision powder X-ray diffractometer with horizontal sample mounts, (Figure 5.38).

The system is based on a dedicated focusing-beam system for general powder and bulk samples. The diffractometer uses a stepping monitor with minimum step angles of 1/10000 degrees to control high-precision horizontal goniometer. The x-ray generator can be controlled across the voltage range of 20.0kV to 50.0kV and current range of 2.0mA to 60.0mA.

This equipment features a 3-kW X-ray generator with a moulded power supply unit, a high-precision O-O wide-angle horizontal goniometer and "Cross Beam Optics", which allows easy, one-touch switching of complex geometry systems.

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Figure 5.38 Theta-Theta Type X-ray Diffractomator Ultima IV Rigaku Corporation.

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5.5   STRUCTURAL CONTROL BY STATIC AND DYNAMIC MONITORING

5.5.1 Static monitoring

Where an important crack pattern is detected and its progressive growth is suspected due to soil settlements, temperature variations or to excessive loads, the measure of displacements in the structure as function of time have to be collected. Monitoring systems can be installed on the structure in order to follow this evolution; in some cases the knowledge of the crack pattern evolution can help preventing the collapse of the structure, (Binda 1997a).

This type of survey is frequently applied to important constructions, like bell towers, (e.g. to the five remaining Pavia towers after the collapse of the Civic Tower in 1989 see P.P Rossi lecture, to the Pisa leaning Tower, to the Dome of the Florence Cathedral in Italy, Figure 5.39), or cathedrals and the system may stay in place for years before a decision can be taken for repair or strengthening.
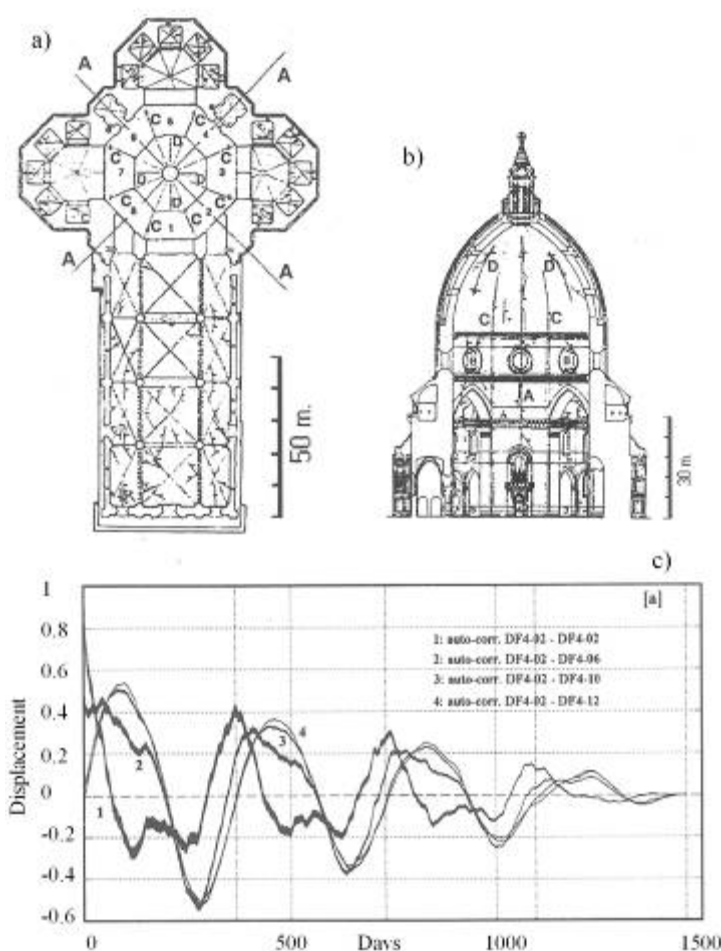
Figure 5.39 S. Maria del Fiore, Florence, Italy: a) position of the major cracks denominated A,B,C,D on the dome; plan; b) position of the major cracks denominated A,B,C,D on the dome, (Chiarugi 1993); c) curves displacement-time obtained as a correlation of the monitored, (Bartoli 1992).

Very simple monitoring systems can be also applied to some of the most important cracks in masonry walls, were the opening of the cracks along the time can be measured by removable extensometers with high resolution. This simple system can give very important information to the designer on the persistence of settlements, etc… Also in this case the monitoring should be a long term one, not less than one year and a half, in order to rule out the influence of temperature variation at every reading of the eventual displacements.

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5.5.2 Dynamic testing and monitoring

Ambient vibration testing (AVT) and operational modal analysis (OMA) of historic towers and monuments is a rather recent topic and only a limited number of full-scale tests is reported in the literature, (Jaishi 2003), (Bennati 2005), (Gentile 2007). Ambient vibration modal testing and analysis seems especially suitable for historic masonry towers and structures, for several reasons: (a) the easy and non destructive way of testing, performed by measuring only the structural response under ambient excitation; (b) the sustainability of testing, that does not interfere with the normal use of the structure and does not induce additional overloads rather than those due to "normal condition", (dead loads, wind and micro-tremors); (c) the multiple-input nature and the wide-band frequency content of ambient excitation, ensuring that the response includes the contribution of a significant number of normal modes; (d) the large number of highly sensitive (10V/g) piezoelectric or force-balanced accelerometers available on the market and relatively inexpensive; (e) the increasing availability of data acquisition and storage systems, that are fully computer-based; (f) the large number of output-only modal identification techniques available in the literature, ranging from the simple Peak Picking technique, (Bendat 1993) to the more advanced Frequency Domain Decomposition, (Brincker 2000) and Stochastic Subspace Identification, (Van Overschee 1996).

The first test carried out could be seen also as the starting one of a periodical survey using vibration monitoring inside a global preventive maintenance programme. Acceptance of vibration monitoring as an effective technique of diagnosis has been supported by different studies, (Niederwanger 1997), (Rossi 1997). These tests are very important to detect eventual anomalies in the diagnosis phase and to calibrate efficient analytic models (FEM), (Gentile 2007), (Gentile 2010). In this way it is possible to verify the effectiveness of the computational methods used in the analyse and control of the structure. The availability of an efficient numerical model allows for checking and predicting the structure behavior to dynamic actions like, for example, winds effects and seismic actions.

The testing procedure could be direct to the updating of a preliminary analysis of the structure by FE modelling with the information gathered from the field tests. The result could be an FE model of the structure which is as closer as possible to reality. This model could then be used to analytically determine:
* the system's response to static or dynamic loads within the limits of linearity;
* the consequences of structural changes like crack pattern propagation as continuous damage or strengthening and repair;
* any changes to the dynamic properties which have occurred in the time elapsed between two measurements.

The key to this vibration analysis/preventive maintenance programme is a systematic, scheduled check of the structures, before, during and after the repair phases. The analysis should be able to assess the condition and the general trend of the structural behavior and advise, for instance, whether there could be a change of the controlled parameters. Results are then compared to the original records from which any long-term change in the structure can be observed. Since only long-term trends are being monitored, subsidiary effects, (e.g. temperature effects), are not considered to affect the results significantly.

The investigation therefore could involve the use of environmental vibration or forced vibration and include a systematic vibration recording and comparison of the analysed data to the model results.

The environmental excitation sources could be the wind, the traffic or the bell ringing in the particular case of towers, (Gentile 2007). The forced vibrations could be produced by local hammering systems or by the use of vibrodines. It has been known for several decades that the frequencies and mode shapes obtained form environment tests are generally in good agreement with results of forced vibration tests.

An accelerometer net is installed in chosen significant parts of the structure. Spectral analysis can be used to extract modal parameters from vibration data. The frequency-domain technique involves

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frequency analysis of the vibration signal and further processing of the resulting spectrum to obtain clearly defined information. With the modal analysis the vibration response consists of summing up the contribution of the infinite number of natural modes, each multiplied by a function of time; the normal modes, detected from the vibration tests analysis are functions of the system properties and the boundary conditions only.

The vibration tests allow detecting the frequencies, the modal shapes and the correspondent modal damping of a structure. Spectra analysis provides a frequency domain resolution of these component physical relationships. These parameters are characteristics of the local and global behavior of a structure. They could be used both to verify the results of a theoretical or numerical model and to monitor the behavior throughout time.

The modal frequencies are parameters representative of the global behavior of the system, while the modal shapes allow detecting the local performance. The analysis of the modal shapes could localise eventually damaged areas. The analysis of the sets of modal shapes deriving from different tests could be done following 2 global criterions:

1. _"Modal Assurance Criterion"_ (MAC), (Allemang 1983), that is a measure of the correspondence of two vectors of modal shapes. The values of MAC equal to 1 show a perfect correlation, while equal to 0 shows an independence between the set of data.
2. _"Coordinate Modal Assurance Criterion"_ (COMAC). The correspondence between the modes is evaluated in a defined position by the index COMAC. It is a measure of the correlation between all the displacements in a point, concerning all the set of corresponding modes. Values of COMAC equal to 1 indicate a good agreement of the modal shapes in the choosen point.

There are two classes of spectral data: autospectrum, (power spectrum), and cross spectrum. The last one could be measured in the frequency domain of the similarity of two functions. Modal frequencies are identified by the locations of peaks in the auto-spectra (ASD) and cross-spectra (CSD); mode shapes are estimated using the ratios of, (square-root ASD), amplitudes and cross-spectral phases are used to determine directions of relative motion. For each natural frequency, mode shapes were normalised to the maximum modal value, (amplitude 1, phase 0). Coherence values are computed to examine the influence of noise and non-linear response of the structure in each mode.

Magnitudes of the coherence function range from 0 to 1 and denoted on how well 2 signals at each frequency are related. Both the autospectrum and the coherence function are used to analyse vibrations and identify their possible sources by measuring the spectra at one point and its coherence value. The result is independent of the power levels at the two points and the transmission gain between them. The coherence function examines the phase difference between auto and cross spectral values over several spectrum measurements. Thus it can detect frequency differences so small to be only variation of the phase. A unit value of the coherence function means that the spectral line at the monitored point is completely coherent with the measured source. The use of this parameter allows detecting torsional modes.

Dynamic testing by forced vibration technique using vibrodine and more recently by ambient vibration technique was used to analyse the global structural condition of several towers. The results together with those supplied by other tests like flat-jack tests and NDT are intended to be used for detection of extended anomalies in the structural behavior. In some cases, ambient vibration modal testing and analysis can possibly help also to limit the number of on-site and laboratory tests of materials that are time-consuming and cost-ineffective. It has to be underlined that data collection of the structure including the historical information, geometrical and topographic survey, survey of damage and crack patterns and the mechanical characterization of materials by ND tests is crucial for assessment, diagnosis and preservation of historic structures and its substitution with (global) full-scale test can be no longer proposed.

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6 NUMERICAL ASSESSMENT OF MASONRY STRUCTURES

6.1 ANALYSIS METHODS - INTRODUCTION

Initially the structures began to be designed using simple rules of thumb based on the workers experience. This method, although quite basic, was used in the construction of big and important structures such as bridges and cathedrals, which today represent an important part of the world heritage. After this, static graphics started being used; it is a quite simple method, which allows solving graphically the structural problems, (Lourenço 2001).

Later, methods based on the concept of limit analysis began to be used. These methods assume that a structure is collapsing and compares the state of collapse with the actual condition of the structure, thus defining its structural safety. This type of analysis has as main advantages, combining the knowledge on the mechanisms and failure loads, with the simplified implementation on practical computational tools, reducing to a minimum the number of required material parameters, which is convenient as this type of parameters have a high degree of uncertainty, and on which it is very difficult to find reliable information, (Orduña et al. 2001).

Only recently, due to the high (and ever increasing) capacity of the modern computers to solve numerical problems, it became possible to simulate the response of materials, such as masonry, (characteristic of historical structures), considering their nonlinear structural behavior. Several methods such as finite elements were then implemented. The basic unit that characterizes this method, the finite element, usually does not represent a structural element but rather a sub-part. Thus, the finite element method can be applied to simulate separately the behavior of various materials, of the elements that compose the material (assembly), of the structure (micro-Modelling) or to simulate in a homogenized and continuous way the global behavior of a composite material (macro-Modelling).

Other Modelling methods also widely used are the Structural Element Models (SEM), the Discrete Element Method (DEM) and its new formulation, the discontinuous deformation analysis (DDA). These last two methods are widely used on the resolution of rock mechanics, which in many cases is quite similar to the problems of masonry. These are also quite useful on the analysis of the failure mechanisms of masonry structures.

6.2 RIGID BLOCK ANALYSIS

Since the first on-site survey campaigns following the significant seismic events occurred in the last decades, several studies were carried out in Italy with the aim of better understanding the failure modalities in the damaged buildings. The direct observation of the failure modes usually recurring in masonry buildings showing similar characteristics enabled to hypothesize a set of failure mechanisms evolving from the first crack appearance up to the total collapse of the assembly of rigid bodies the building is transformed into after being cracked. In most cases, the damage observed indicated a local failure of some building portions, which however did not caused the global collapse of the structure: as illustrated in Deliverable 3.1 (Inventory of earthquake-induced failure mechanisms related to construction types, structural elements, and materials), in these buildings, in fact, the absence of systematic connections between intersecting walls and between walls and horizontal structures may cause kinematic mechanisms related to the loss of equilibrium of structural portions rather than to states of stress exceeding the materials ultimate capacity, (Giuffré et al., 1999).

Based on the observation of real seismic failure modes of historical and traditional buildings in Italy, Giuffré (1991, 1995), see also Giuffrè and Carocci (Giuffrè 1993a,b) and Carocci (2001), proposed an approach for the study of the seismic vulnerability of masonry buildings based on their decomposition into rigid blocks, (Figure 6.1 and Figure 6.2). The collapse mechanisms are then analyzed by applying kinematic limit analysis. This approach is particularly interesting as a tool for seismic analysis of buildings which do not conform to box behavior due to the lack of stiff floor slabs or because of weaker partial collapses affecting façade or inner walls.

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Damage to masonry buildings can be essentially interpreted on the basis of two fundamental collapse mechanisms. According Giuffrè definition, (Giuffrè 1993a), the "First Damage Mode" is produced by seismic actions perpendicular to the wall (out-of-plane) that cause the overturning of the whole wall panel or of a significant portion of it, (Figure 6.1A). A signature of such damage, short of collapse, can be the shedding of a portion of the exterior leaf of masonry. Another can be the formation of vertical cracks at the corners of a building where the wall began to form a hinge from the swaying.

This behavior represents the highest building vulnerability and in the past was prevented by the use of ties to compensate for the lack of connection between the external walls and the ones orthogonal to them. The effectiveness of the tie consists in involving the walls orthogonal to the facade as containing elements. They resist to the seismic action transmitted by the facades as in plane action and exert a higher resistance towards such action. When the action overcomes the strength, also the walls stressed in their plane can crack, according to the classic diagonal course which isolates a triangular part of the wind-brace wall and makes it participate to the cracking motion. This further damage modality - called "Second Mode of Damage" - can be checked only when the "first mode" doesn't occur thanks to metallic connections, (Figure 6.1B).

Figure 6.1 Failure modes for buildings with no ties (A) and with ties (B) anchoring the façade to lateral walls, (Carocci, 2001).
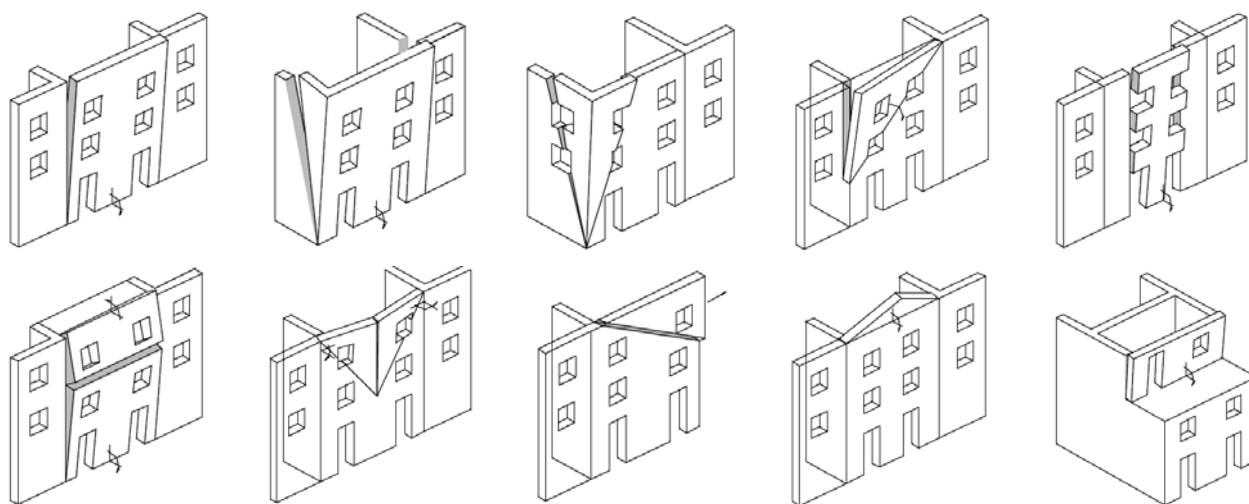
Figure 6.2 Failure mechanisms for buildings embedded within urban texture, (based on D'Ayala and Speranza, 2002a).

While the "first mode" is always ruinous, as it implies the complete collapse of the wall and consequent ruin of all supported elements, the "second mode" does not necessarily determine the collapse, though it still implies small, medium and even large cracks of the wind-brace walls.

The Second Damage Mode is caused, as said before, by forces acting in the plane of the wall and is usually marked by inclined cracks associated with shear forces that often result in an "X" pattern,

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but it seldom reaches the total collapse. However, when a full shear crack occurs during an earthquake, the triangular sections of the panel can become unstable, leading to collapse.

In historic centres, as well as in building evolved in time the addition of adjacent constructions or portions implies the lack of strong connections between the parts. The "consequence of this organic defect is the particular fragility of the historical house towards the seismic action", (Giuffrè 1993), (Giuffrè 1996).

The data surveyed allowed to create several collections, (called abaci), in which the principal failure modalities for the different structural typologies, (buildings, churches, etc…), were reported in the form of interpretative graphical schemes, involving one or more macro-elements, (see Annex 1 of the Deliverable 3.1 - damage abacus).

Macro-elements are defined by single or combined structural components, (walls, floors, roof and their sub-assemblages), considering their mutual bond (potential damage pattern, cracks, borders of poor connections, etc...) and restraints, (e.g. the presence of ties or ring beams), the constructive deficiencies and the characteristics of the constitutive materials, (Giuffrè 1993; Doglioni 1994). The decomposition of the building into sub portions derives from the impossibility to define particular correlations to interpret the overall behavior of the structure. In fact, macro-elements behave independently as a whole and present a mutual interaction with visible cracks in correspondence of their contact surfaces or influence area: such boundary regions are characterized by scarce connections or damage patterns previously occurred between the facing macro-elements. Thus they are elements in hazardous conditions for possible incipient brittle collapse following kinematic mechanisms (Borri et al. 1999). The previously mentioned abaci allowed a systematization of the mechanical models able to describe the behavior of macro-elements by kinematic models

The macro-elements approach for the structural evaluation of the seismic behavior consists in the use of local simplified models, (kinematic mechanisms), based on limit analysis and applied to single structural elements, rather than in the conventional evaluation of the overall structural behavior; however, this approach can lead to a global evaluation of the structure, by successive application of the method to the different macro-elements that compose the building. The reliability of the macro-element approach for describing the real structural behavior of existing masonry structures than common procedures, based on the "box" behavior of the structure and on the elasto-plastic behavior of the masonry, has been confirmed by several studies (Penazzi et al. 2000; Valluzzi et al. 2001).

In particular, two main collapse mechanisms were considered, named mode I failure, (out-of-plane masonry wall failure), and mode II failure, (in-plane masonry wall failure): not uncommon is the occurrence of combined principal failure modes in consequence of the arbitrariness of the direction of the seismic action:

- Out-of-plane mechanisms are the most dangerous and involve walls subjected to horizontal actions orthogonal to their plane. Their partial or total overturning is the main result, which is counteracted by the possible presence of connection elements, (ties, ring beams), or intrinsic resisting effects (e.g. arch effect of the wall in its thickness). The proposed analysis method is based on equilibrium equations which can take into account also the strength of the materials, (as well crushing of masonry, tension in the tie, etc…);

- In-plane mechanisms relate to walls parallel to the seismic action. The relative damage (shear cracks), generally does not lead the structure to collapse, in comparison with the out-of-plane mechanisms. Kinematics chains describe the in-plane rigid rotation of the resisting structural portions of the building, defined by particular geometrical, (dimensions of septa, openings), and bond conditions, (connections, presence of ties), subjected to in-plane horizontal actions.

The procedure for the quantitative evaluation of kinematic models consists in the identification of the values of horizontal static-equivalent forces, (and therefore of the values of the masses accelerations), that can activate specific mechanisms of local failure / overturning of structural

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macro-elements. Once the critical structural configuration is defined, the subsequent step is the identification of the most probable collapse mechanisms of each macro-element. Kinematic models provide a coefficient $c = a/g$ (where a is the mean response acceleration and g the gravity acceleration), which represents the seismic masses multiplier characterizing the limit of the equilibrium conditions for the considered element: in fact, under the hypothesis of the procedure, the ultimate capacity of the building depends on the stability of its macro-elements. In simplified assessment procedures, the mechanism connected to the lowest value of c is the weakest one and, consequently, the most probable to occur: in-plane mechanisms are characterized by c coefficients higher than the out-of-plane ones, (Borri et al. 1999).

This limit analysis approach depends on few geometric and mechanical parameters and therefore it does not require an extremely accurate survey and time-consuming computation. In general, in order to calculate the c coefficient, different variables are necessary for each mechanism: for every wall at every floor of the building it is necessary to know some basic data, as height (h), length (l), thickness (b or s), height of eventual holes (hf), type of material, (masonry properties), acting load (N) and arm of the load regarding the external edge of the wall on which it is applied (d).

Since the 80's in Italy, empirical evaluations, by the so called "vulnerability indexes", (particularly for masonry buildings), (Benedetti 1984), have been proposed, based on weighted sums of vulnerability factors, related both to structural irregularity aspects recorded by rapid systematic or sample surveys and to the actual calculations of the resistance to horizontal actions of the masonry walls. The aim was to compare the vulnerability of different buildings, (and thus the priorities for strengthening operations), and to provide damage scenarios for different seismic intensities.

Within these methodologies and with specific reference to historic masonry buildings, some procedures have been proposed: they are based on the identification of the values of horizontal static-equivalent forces, (and therefore of the values of the masses accelerations), that can activate specific mechanisms of local failure / overturning of structural macro-elements, (composed by single walls or sub-assemblages, as intersecting walls, walls and floors or roof, etc…), in-plane and, especially, out-of-plane, (Bernardini 1988), (Bernardini 1990). In these buildings, in fact, the absence of systematic connections between intersecting walls and between walls and horizontal structures may cause kinematic mechanisms related to the loss of equilibrium of structural portions rather than to states of stress exceeding the materials ultimate capacity, (Giuffrè 1999); this limit analysis approach depends on few geometrical and mechanical parameters and therefore it does not require an extremely accurate survey and time-consuming computation, (Bernardini 1999).

Once the critical structural configuration is defined, the subsequent step is the identification of the most probable collapse mechanisms of each macro-element. The studies based on on-site surveys after seismic events allowed to create abaci of the typical damages occurring in constructive typologies such as buildings, churches, palaces, (BBCC 1997, 2006), which led to a consequent systematization of the mechanical models able to describe their behavior by kinematic models, (Bernardini 1988) (Bernardini 1990), (Borri 1999a,b), (D'Ayala 2003).

Kinematic models provide the previously defined coefficient c, which represents the seismic masses multiplier characterizing the limit of the equilibrium conditions for the considered element. In simplified assessment procedures, the mechanism connected to the lowest value of c is the weakest one and, consequently, the most probable to occur: in-plane mechanisms are characterized by c coefficients higher than the out-of-plane ones, (Bernardini 1990), (Giuffrè 1999).

The new methodology for the damage assessment considers the most significant collapse mechanisms in each architectonic part of the building, (Figure 6.3 and Figure 6.4).

Several works on seismic vulnerability evaluation of masonry structures through limit analysis procedures have been proposed, (Bernardini 1988), (Bernardini 1990), (Bernardini 1999), (Giuffrè 1993a,b), (Giuffrè 1997), (Brun 1999), (de Felice 2001), (D'Ayala 1999a), (D'Ayala 2003a), (Modena 2004), (Munari 2009), but the research was limited to the calculation of the seismic activation multiplier, even if evaluated for complex mechanisms. This approach of limit analysis applied to existing masonry buildings in seismic areas is now provided by the updated Italian seismic code, (PCM 2003), (PCM 2005), which finally takes into account the high vulnerability of existing masonry buildings not satisfying assumptions commonly more suitable for new

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earthquake-proof structures. In this field, another important document is represented by the Guidelines published by the Italian Ministry of Cultural Heritage for the evaluation and mitigation of seismic risk of the architectural heritage, (BBCC 1997, 2006), (Moro 2007).
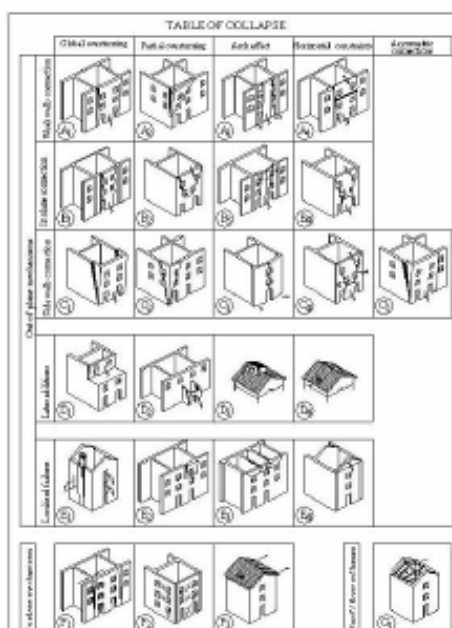
Figure 6.3 Damage mechanisms for row buildings, (D'Ayala 1999a).
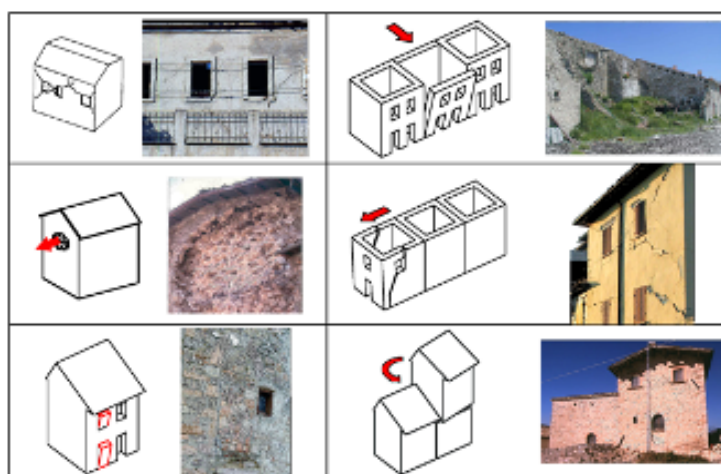
Figure 6.4 Example of damage mechanisms, (Penazzi 2000).

More recently, Giuffré's proposal has experienced renewed interest thanks to the possibility of combining block analysis with the capacity spectrum method (Fajfar 1999, Lagormarsino et al., 2003, Lagomarsino, 2006) for the seismic assessment of masonry structures. The method is applied to buildings, churches and towers. The resulting verification methodology has been adopted by the seismic Italian code OPCM 3274 (PCM 2003).

This approach of limit analysis applied to existing masonry buildings in seismic areas is now provided by the updated Italian seismic code (N.T.C. 2008; O.P.C.M. 3274/2003 (PCM 2003); O.P.C.M. 3431/2005 (PCM 2005)), which finally takes into account the high vulnerability of existing masonry buildings not satisfying assumptions commonly more suitable for new earthquake-proof structures.

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Research on the possibilities of classical limit analysis is still being carried out. Recently, improved graphic oriented techniques for analysis of masonry arch and vault structures, based on the combination of the static and kinematic approaches, have been presented by Block et al. (2006). Limit analysis is currently exploited as a useful tool to analyze ancient structures, (Ochsendorf, J, 2002, De Luca et al., 2004, Block 2005). Roca et al. (2007) have produced a method for graphically-oriented analysis of reinforced masonry structures based on the same principles.

Most modern computer developments based on limit analysis exploit the potential of the kinematic approach for the analysis of masonry structures composed of block assemblages. The following hypotheses are normally adopted: (1) Limit load occurs at small overall displacements. (2) Masonry has zero tensile strength. (3) Shear failure at the joints is perfectly plastic. (4) Hinging failure mode at a joint occurs for a compressive load independent from the rotation. Hypothesis (1) is true for most cases. Assumption (3) is fully supported by experimental results. In the case of masonry crushing, hypothesis (4) might be questionable, but crushing behavior has minor importance in the response of masonry structures except for very shallow arches, pillars, towers and massive vertical structures.

The use of the aforementioned principles in combination with modern computers and advanced numerical methods has provided powerful tools for the analysis of masonry constructions. Livesley, (1978), Melbourne and Gilbert (1994), (see also Gilbert and Melbourne, 1994, and Gilbert, 2007), Baggio and Trovalusci (1998), Ferris and Tin-Loi (2001), Casapulla and D'Ayala (2001), Orduña y Lourenço (2003, 2005a, 2005b) and Gilbert et al. (2006), among others, have proposed different methods for the assessment of masonry structures by limit analysis. Modern developments afford the analysis of blocky structures for both in-plane and out-of-plane loading.

Orduña and Lourenço (2003, 2005a, 2005b) have proposed a Cap Model for limit analysis for both plane or spatial structures made of rigid blocks which takes into account the non-associated flow rules and limited compressive strength of masonry. Results obtained with the model have been satisfactorily compared with available experimental results. An example concerning a simple wall subjected to seismic loads is given in Figure 6.5 where the amount of horizontal forces resisted is measured as a multiplier α over the gravity forces.
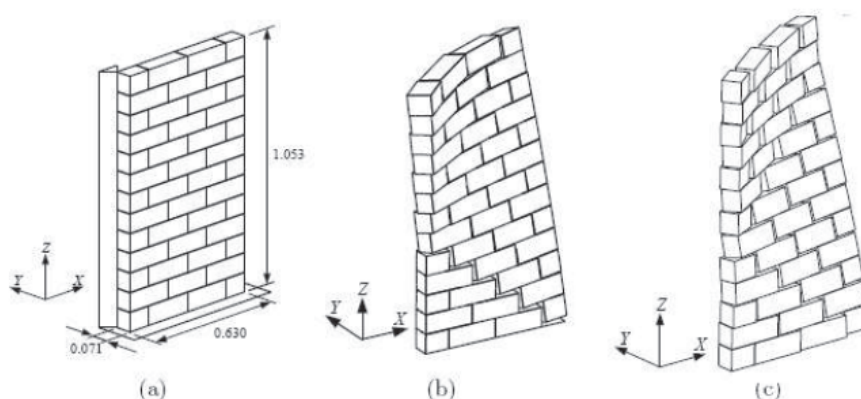
Figure 6.5 Out-of-plane loaded wall, supported at one edge; (a) model; (b) FEM failure mechanism (α = 0.210); (c) limit analysis failure mechanism (α= 0.216), (Orduña and Lourenço 2005b).

In the following paragraphs, some typical out-of plane and in-plane kinematic mechanisms are analyzed: for each mechanism a short description, a graphical scheme, the definition of the involved variables, the formulas for the determination of the *c* coefficient and some additional notes are reported.

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6.2.1 Out of plane mechanisms

6.2.1.1 Mechanisms associated to vertical stripes of masonry

OVERTURNING OF A MONOLITHIC WALL SIMPLY SUPPORTED BY THE ORTHOGONAL WALL (Mechanism 2.2.1 - Annex 1 Deliverable 3.1)

Considering the seismic action as an equivalent static force (product of the masses of the wall for seismic acceleration, assumed constant along the height of the wall), the wall will then be subject to the action of an overturning moment (given by the seismic action applied at the center of gravity of the considered masonry wall by its arm h_1) which will tend to rotate the wall around the cylindrical hinge that is formed at its base (point A).

An opposing force generated by the weight of the wall and by the loads imposed on it will counter this action. It is then possible to determine the masses that activate the mechanism (c coefficient) through the imposition of equilibrium conditions to the rotation.

Scheme of the kinematic mechanism

Table 6.1 Variables
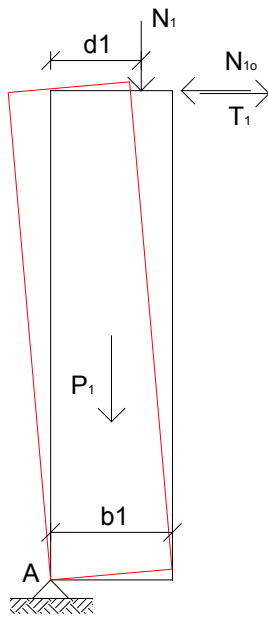
	Variables
P_1	Self weight of the wall
h_1	Height of the wall
b_1	Average thickness of the wall
N_1	Load acting on the wall
d_1	Arm of the load
N_{1o}	Horizontal component of the vault thrust
T_1	Tension in the tie

Stabilizing moment: $M_S = P_1 \dfrac{b_1}{2} + N_1 d_1$ Overturning moment: $M_R = c \cdot P_1 \dfrac{h_1}{2} + c \cdot N_1 \cdot h_1 + N_{1o} \cdot h_1$

$$c = \frac{P_1 \dfrac{b_1}{2} + N_1 d_1 + \left(T_1 - N_{1o}\right) h_1}{P_1 \dfrac{h_1}{2} + N_1 h_1}$$

Tension in the tie T_1 $T_1 = a \cdot \left(\dfrac{P_1}{N_1} + N_1\right) - \left[\dfrac{P_1 \dfrac{b_1}{2} + N_1 d_1}{h_1}\right]$

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OVERTURNING OF A DOUBLE-LAYER WALL SIMPLY SUPPORTED BY THE PERPENDICULAR WALL (Mechanism 2.2.3 - Annex 1 Deliverable 3.1)

The wall is considered as composed by a "double layer" texture. Under the seismic action may thus arise an "in parallel" behavior of the two layers that form the wall, which do not react to the horizontal action as a single body, but as two independent panels that rotate each around its base hinge. The wall shall be considered not connected to any perpendicular wall.

Scheme of the kinematic mechanism

Table 6.2 Variables
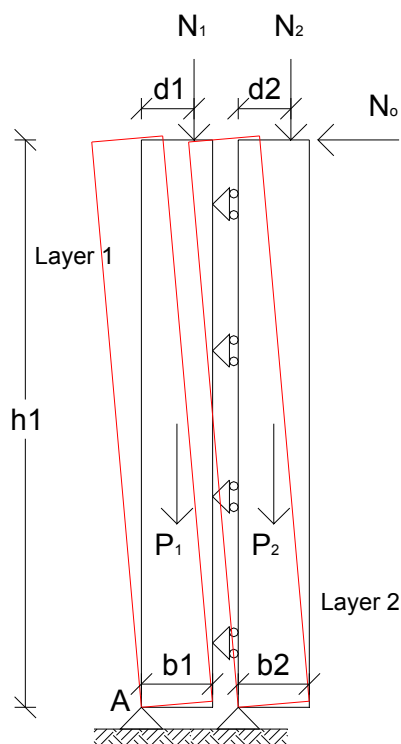
	Variables
P_1	Self weight of the external layer of the wall
h_1	Height of the wall
b_1	Average thickness of the external layer of the wall
N_1	Load acting on the external layer of the wall
N_o	Horizontal component of the vault thrust
d_1	Arm of the load
P_2	Self weight of the internal layer of the wall
h_2	Height of the wall ($h_2 = h_1$)
b_2	Average thickness of the internal layer of the wall
N_2	Load acting on the internal layer of the wall
d_2	Arm of the load

Stabilizing moment: $M_S = P_1 \dfrac{b_1}{2} + N_1 d_1$

Overturning moment: $M_R = c \cdot (P_1 + P_2)\dfrac{h_1}{2} + c \cdot (N_1 + N_2) \cdot h_1 + N_o \cdot h_1$

$$c = \frac{P_1 \dfrac{b_1}{2} + N_1 d_1 - N_o h_1}{(P_1 + P_2)\dfrac{h_1}{2} + (N_1 + N_2)h_1}$$

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OVERTURNING OF A WALL CONNECTED TO A PERPENDICULAR WEAK WALL (Mechanism 2.2.5 - Annex 1 Deliverable 3.1)

In many cases, the angles have adequate connections between the walls that meet in the node. These masonries are usually constructed at the same time or have been consolidated connection the orthogonal wall panels. In this case the wall that suffers the out-of-plane mechanism rotates around the cylindrical hinge that is formed at the base carrying a wedge-shaped portion of the orthogonal wall.

The formation of the wedge on the orthogonal wall is strongly influenced by the presence of openings and by the texture of the wall.

For orthogonal wall without openings, the angle formed by the diagonal of the wedge increases with the average size of orthostats (elements placed with the longer side in the plan of the wall). Limit values of 15° for walls built with small stones and poor mortar and 30° for brick walls may be used.

Scheme of the kinematic mechanism

Table 6.3 Variables
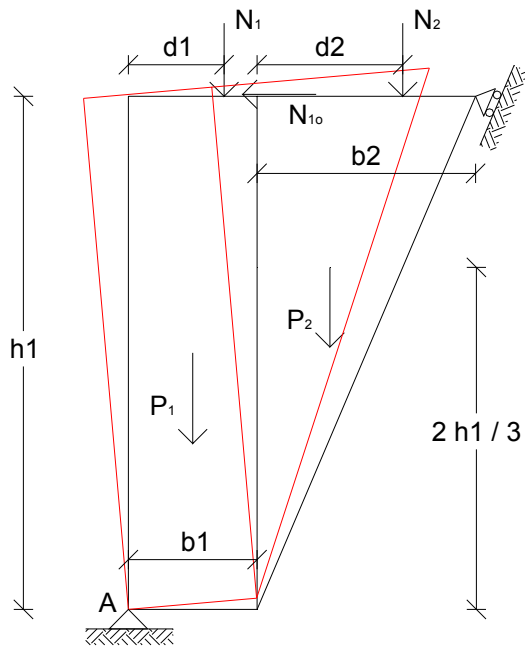
	Variables
P_1	Self weight of the wall
h_1	Height of the wall
b_1	Average thickness of the wall
N_1	Load acting on the wall
N_{1o}	Horizontal component of the vault thrust
d_1	Arm of the load
P_2	Self weight of the detached wedge
h_2	Height ($h_2 = h_1$)
b_2	Length of the base of the detached wedge
N_2	Load acting on the detached wedge
d_2	Arm of the load

Stabilizing moment: $M_S = P_1 \dfrac{b_1}{2} + P_2 \left(b_1 + \dfrac{b_2}{3} \right) + N_1 \cdot d_1 + N_2 \left(b_1 + d_2 \right)$

Overturning moment: $M_R = c \cdot P_1 \dfrac{h_1}{2} + c \cdot \left(N_1 + N_2 \right) \cdot h_1 + c \cdot P_2 \dfrac{2}{3} h_1 + N_{1o} \cdot h_1$

$$c = \dfrac{P_1 \dfrac{b_1}{2} + P_2 \left(b_1 + \dfrac{b_2}{3} \right) + N_1 d_1 + N_2 \left(b_1 + d_2 \right) - N_{1o} h_1}{P_1 \dfrac{h_1}{2} + P_2 \dfrac{2}{3} h_1 + \left(N_1 + N_2 \right) h_1}$$

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OVERTURNING OF A WALL RESTRAINED AT THE TOP BY A TIE (Mechanism 2.2.6 - Annex 1 Deliverable 3.1)

If the wall is retained at its top by a tie to prevent the overturning, the wall does not reach the collapse until the resultant of the loads falls within the thickness of the wall; conversely, when the resultant touches the outer edge of the wall at that point a hinge is activated and the wall reaches the collapse. The problem is to identify the height at which the hinge is formed: it is resolved through the application of the principle of virtual work and searching of the minimum value c for the activation of the mechanism.

It is more probable that the mechanism will occur at the top floor, but it can also be applied to the walls of the other levels.

T_1 is considered in the mechanism and N_{1o} (horizontal thrust of the vault) is not considered (it is considered completely compensated by the containment of the tie).

Scheme of the kinematic mechanism

Table 6.4 Variables
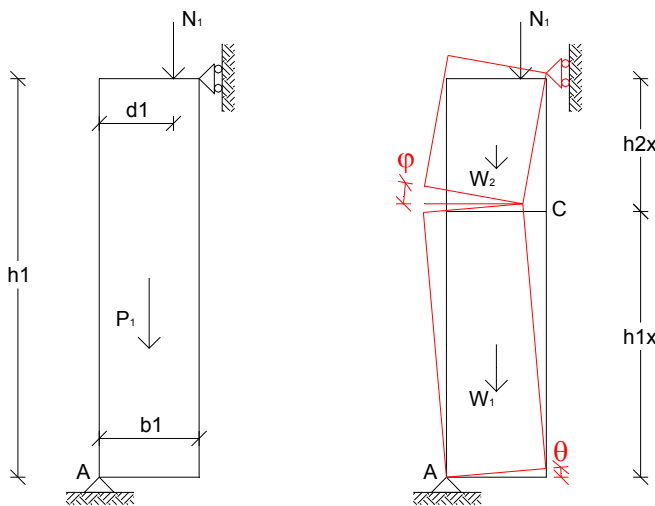
	Variables
P_1	Self weight of the wall
h_1	Height of the wall
b_1	Average thickness of the wall
N_1	Load acting on the wall
d_1	Arm of the load ($d_1 = b_1/2$)

Principle of virtual work: $-W_1\delta_{1_y} - W_2\delta_{2_y} - N\delta_{N_y} + cW_1\delta_{1_x} + cW_2\delta_{2_x} + cN\delta_{N_x} = 0$

$$c = \frac{b_1}{h_1} \cdot \frac{2x + \frac{N_1}{P_1}(x+1)x}{x-1}$$

where: $h_{1x} = \frac{x-1}{x} \cdot h_1$ $\qquad$ $h_{2x} = \frac{1}{x} \cdot h_1$

Rupture height: $\qquad \frac{dc}{dx} = 0 \implies x = 1 + \sqrt{2\frac{N_1 + P_1}{N_1}}$

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DIFFERENT HEIGHT OF THE FLOORS (Mechanism 5.2.3 - Annex 1 Deliverable 3.1)

In many situations masonry walls have to withstand bending stresses induced by the earthquake and due to the non-alignment of floors: similar patterns can also be found in slender wall panels, as sometimes happens in stairwells, in the gables of roofs (where beams often rely on walls with small thickness) or in spaces where orthogonal septa were removed.

Scheme of the kinematic mechanism

Table 6.5 Variables
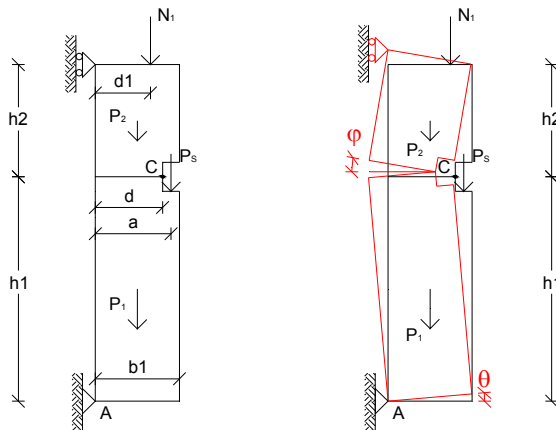
	Variables
P_1	Self weight of the lower wall
h_1	Height of the lower wall
b_1	Average thickness of the wall
P_2	Self weight of the upper wall
h_2	Height of the upper wall
b_2	Average thickness of the wall ($b_2 = b_1$)
N_1	Load acting on the wall
d_1	Arm of the load ($d_1 = b_1/2$)
P_s	Load of the intermediate floor
a	Arm of the load P_s
d	Position of the C hinge

Principle of virtual work: $-P_1\delta_{P_{1_y}} - P_S\delta_{P_{S_y}} - P_2\delta_{P_{2_y}} - N_1\delta_{N_{1_y}} + cP_1\delta_{P_{1_x}} + cP_S\delta_{P_{S_x}} + cP_2\delta_{P_{2_x}} + cN_1\delta_{N_{1_x}} = 0$

$$c = \frac{P_1\dfrac{b_1}{2} + (P_2 + N_1)d + P_S a + \dfrac{P_2 + N_1}{h_2}\left(d - \dfrac{b_1}{2}\right)h_1}{\dfrac{(P_1 + P_2 + 2P_S)h_1}{2}}$$

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OVERTURNING OF A WALL RESTRAINED AT THE TOP BY A RING BEAM (Mechanism 2.2.4 - Annex 1 Deliverable 3.1)

The mechanism takes into account the case of a wall supporting the load of the upper levels, restrained at the top by a reinforced concrete ring beam: the ring beam apply an effective containment function (perpendicular ties that restrain the ring beam are present or the ring beam is present around the entire building with a full ring), but a perfect connection existing between the beam and the masonry is not considered. The containment action is then transmitted to the wall through the friction between masonry and the ring beam: the overturning mechanism is then countered by friction exercised by the ring beam.

The horizontal thrust resulting from the presence of any vault is considered to be fully compensated by the containment action of the ties or of the orthogonal ring beam.

Scheme of the kinematic mechanism

Table 6.6 Variables
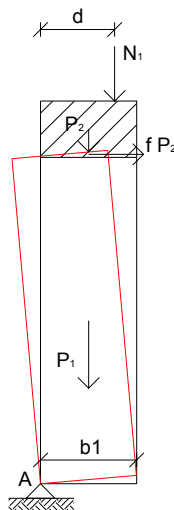
	Variables
P_1	Self weight of the wall
h_1	Height of the wall
b_1	Average thickness of the wall
f	Ring beam – wall fiction coefficient
P_2	Self weight of the ring beam
h_2	Height of the ring beam
b_2	Thickness of the ring beam ($b_2 = b_1 = b$)
N_1	Load acting on the wall
d_1	Arm of the load

Stabilizing moment: $M_S = P_1 \dfrac{b_1}{2} + P_2 \dfrac{b_1}{2} + f \cdot (P_2 + N_1) \cdot h_1 + N_1 d_1$

Overturning moment: $M_R = c \cdot P_1 \dfrac{h_1}{2}$

$$c = \frac{P_1 + P_2}{P_1} \frac{b_1}{h_1} + 2f \frac{P_2 + N_1}{P_1} + 2 \frac{N_1}{P_1} \frac{d_1}{h_1}$$

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OVERTURNING OF MULTI-FLOOR WALLS NOT CONNECTED TO AN ORTHOGONAL WALL
(Mechanism 2.2.8 - Annex 1 Deliverable 3.1)

In the case of multi-storey buildings as many rotation kinematic mechanisms as the number of walls can be activated. The formulas given here refer to the calculation of the stabilizing moment and overturning moment for the case in which all the walls jointly rotate around the hinge at the base of the building.

Scheme of the kinematic mechanism

Table 6.7 Variables

	Variables
P_1	Self weight of the wall 1
h_1	Height of the wall
b_1	Average thickness of the wall
N_1	Load acting on the wall 1
N_{1o}	Horizontal component of the vault thrust
d_1	Arm of the load
P_2	Self weight of the wall 2
h_2	Height of the wall
b_2	Average thickness of the wall
N_2	Load acting on the wall 2
N_{2o}	Horizontal component of the vault thrust
d_2	Arm of the load
P_3	Self weight of the wall 3
h_3	Height of the wall
b_3	Average thickness of the wall
N_3	Load acting on the wall 3
N_{3o}	Horizontal component of the vault thrust
d_3	Arm of the load
P_4	Self weight of the wall 4
h_4	Height of the wall
b_4	Average thickness of the wall
N_4	Load acting on the wall 4
N_{4o}	Horizontal component of the vault thrust
d_4	Arm of the load

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Stabilizing moment:

$$M_S = P_1\left(\frac{b_1}{2} - t_1\right) + N_1(d_1 - t_1) + P_2\left(\frac{b_2}{2} - t_1\right) + N_2(d_2 - t_1) + P_3\left(\frac{b_3}{2} - t_1\right) + N_3(d_3 - t_1) + P_4\left(\frac{b_4}{2} - t_1\right) + N_4(d_4)$$

$$- N_{1o} \cdot h_1 - N_{2o} \cdot (h_1 + h_2) - N_{3o} \cdot (h_1 + h_2 + h_3) - N_{4o} \cdot h_{tot}$$

Overturning moment:

$$M_R = c \cdot P_1 \frac{h_1}{2} + c \cdot N_1 h_1 + c \cdot P_2\left(h_1 + \frac{h_2}{2}\right) + c \cdot N_2(h_1 + h_2) + c \cdot P_3\left(h_1 + h_2 + \frac{h_3}{2}\right) + c \cdot N_3(h_1 + h_2 + h_3)$$

$$+ c \cdot P_4\left(h_1 + h_2 + h_3 + \frac{h_4}{2}\right) + c \cdot N_4 h_{tot}$$

$$c = \frac{P_1\frac{b_1}{2} + N_1 d_1 + P_2\frac{b_2}{2} + N_2 d_2 + P_3\frac{b_3}{2} + N_3 d_3 + P_4\frac{b_4}{2} + N_4 d_4 - N_{1o} \cdot h_1 - N_{2o} \cdot (h_1 + h_2) - N_{3o} \cdot (h_1 + h_2 + h_3) - N_{4o} \cdot h_{tot}}{P_1\frac{h_1}{2} + N_1 h_1 + P_2\left(h_1 + \frac{h_2}{2}\right) + N_2(h_1 + h_2) + P_3\left(h_1 + h_2 + \frac{h_3}{2}\right) + N_3(h_1 + h_2 + h_3) + P_4\left(h_1 + h_2 + h_3 + \frac{h_4}{2}\right) + N_4 h_{tot}}$$

where $h_{tot} = h_1 + h_2 + h_3 + h_4$

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OVERTURNING OF MULTI-FLOOR WALLS NOT CONNECTED TO AN ORTHOGONAL WALL
(Mechanism 2.2.8 - Annex 1 Deliverable 3.1)

To solve the problem of out-of-plane overturning of a multi-storey wall simply supported by the orthogonal walls, the formula for the calculation of the activation coefficient c derived from equilibrium of overturning moment due to seismic action and stabilizing moment due to the weights of the structure and to the action of ties is considered.

The formulas for the calculation of the tension in the ties depend on a predetermined "project" value of seismic acceleration on the building.

Scheme of the kinematic mechanism

Table 6.8 Variables
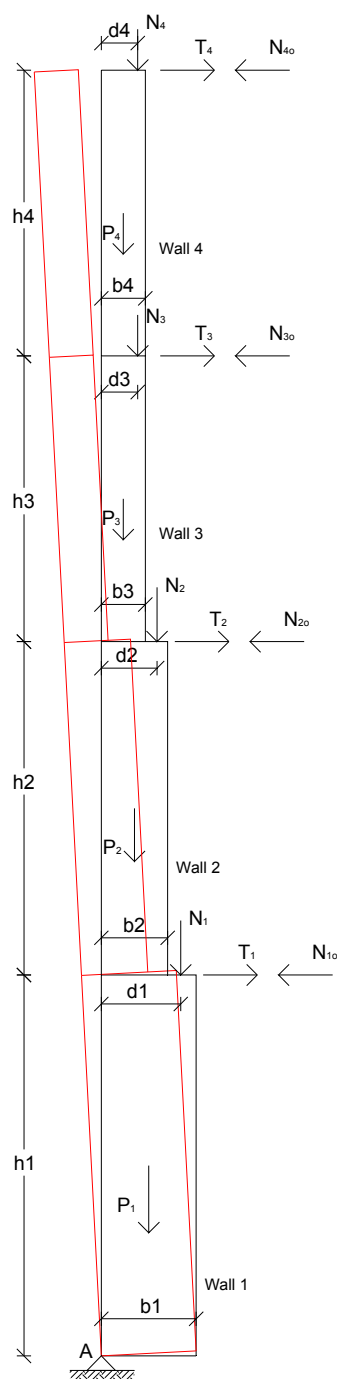
Variables	
P_1	Self weight of the wall 1
h_1	Height of the wall
b_1	Average thickness of the wall
N_1	Load acting on the wall 1
N_{1o}	Horizontal component of the vault thrust
d_1	Arm of the load
T_1	Tension in the tie 1
P_2	Self weight of the wall 2
h_2	Height of the wall
b_2	Average thickness of the wall
N_2	Load acting on the wall 2
N_{2o}	Horizontal component of the vault thrust
d_2	Arm of the load
T_2	Tension in the tie 2
P_3	Self weight of the wall 3
h_3	Height of the wall
b_3	Average thickness of the wall
N_3	Load acting on the wall 3
N_{3o}	Horizontal component of the vault thrust
d_3	Arm of the load
T_3	Tension in the tie 3
P_4	Self weight of the wall 4
h_4	Height of the wall
b_4	Average thickness of the wall
N_4	Load acting on the wall 4
N_{4o}	Horizontal component of the vault thrust
d_4	Arm of the load
T_4	Tension in the tie 4

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Stabilizing moment:

$$M_S = P_1\left(\frac{b_1}{2} - t_1\right) + N_1(d_1 - t_1) + P_2\left(\frac{b_2}{2} - t_1\right) + N_2(d_2 - t_1) + P_3\left(\frac{b_3}{2} - t_1\right) + N_3(d_3 - t_1) + P_4\left(\frac{b_4}{2} - t_1\right) + N_4(d_4)$$

$$+ (T_1 - N_{1o}) \cdot h_1 + (T_2 - N_{2o}) \cdot (h_1 + h_2) + (T_3 - N_{3o}) \cdot (h_1 + h_2 + h_3) + (T_4 - N_{4o}) \cdot h_{tot}$$

Overturning moment:

$$M_R = c \cdot P_1 \frac{h_1}{2} + c \cdot N_1 h_1 + c \cdot P_2\left(h_1 + \frac{h_2}{2}\right) + c \cdot N_2(h_1 + h_2) + c \cdot P_3\left(h_1 + h_2 + \frac{h_3}{2}\right) + c \cdot N_3(h_1 + h_2 + h_3)$$

$$+ c \cdot P_4\left(h_1 + h_2 + h_3 + \frac{h_4}{2}\right) + c \cdot N_4 h_{tot}$$

$$c = \frac{P_1 \frac{b_1}{2} + N_1 d_1 + P_2 \frac{b_2}{2} + N_2 d_2 + P_3 \frac{b_3}{2} + N_3 d_3 + P_4 \frac{b_4}{2} + N_4 d_4 + T_1 \cdot h_1 + T_2 \cdot (h_1 + h_2) + T_3 \cdot (h_1 + h_2 + h_3) + T_4 \cdot h_{tot}}{P_1 \frac{h_1}{2} + N_1 h_1 + P_2\left(h_1 + \frac{h_2}{2}\right) + N_2(h_1 + h_2) + P_3\left(h_1 + h_2 + \frac{h_3}{2}\right) + N_3(h_1 + h_2 + h_3) + P_4\left(h_1 + h_2 + h_3 + \frac{h_4}{2}\right) + N_4 h_{tot}}$$

where $h_{tot} = h_1 + h_2 + h_3$

Tension in the tie T_1:

$$T_1 = a \cdot \frac{1}{h_1}\left[\begin{array}{l} P_1 \frac{h_1}{2} + N_1 h_1 + P_2\left(h_1 + \frac{h_2}{2}\right) + N_2(h_1 + h_2) + \\ + P_3\left(h_1 + h_2 + \frac{h_3}{2}\right) + N_3(h_1 + h_2 + h_3) + \\ + P_4\left(h_1 + h_2 + h_3 + \frac{h_4}{2}\right) + N_4 h_{tot} \end{array}\right] - \frac{1}{h_1}\left[\begin{array}{l} P_1 \frac{b_1}{2} + N_1 d_1 + P_2 \frac{b_2}{2} + N_2 d_2 + T_2 \cdot (h_1 + h_2) + \\ + P_3 \frac{b_3}{2} + N_3 d_3 + T_3 \cdot (h_1 + h_2 + h_3) + \\ + P_4 \frac{b_4}{2} + N_4 d_4 + T_4 h_{tot} \end{array}\right]$$

Tension in the tie T_2:

$$T_2 = a \cdot \frac{1}{h_2}\left[P_2 \frac{h_2}{2} + N_2 h_2 + P_3\left(h_2 + \frac{h_3}{2}\right) + N_3(h_2 + h_3) + P_4\left(h_2 + h_3 + \frac{h_4}{2}\right) + N_4(h_2 + h_3 + h_4)\right]$$

$$- \frac{1}{h_2}\left[P_2 \frac{b_2}{2} + N_2 d_2 + P_3 \frac{b_3}{2} + N_3 d_3 + T_3 \cdot (h_2 + h_3) + P_4 \frac{b_4}{2} + N_4 d_4 + T_4 \cdot (h_2 + h_3 + h_4)\right]$$

Tension in the tie T_3:

$$T_3 = a \cdot \frac{1}{h_3}\left[P_3 \frac{h_3}{2} + N_3 h_3 + P_4\left(h_3 + \frac{h_4}{2}\right) + N_4(h_3 + h_4)\right] - \frac{1}{h_3}\left[P_3 \frac{b_3}{2} + N_3 d_3 + P_4 \frac{b_4}{2} + N_4 d_4 + T_4 \cdot (h_3 + h_4)\right]$$

Tension in the tie T_4:

$$T_4 = a \cdot \left(\frac{P_4}{2} + N_4\right) - \frac{1}{h_4}\left[P_4 \frac{b_4}{2} + N_4 d_4\right]$$

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MASONRY WALL WITH SOME CONNECTION TO THE ORTHOGONAL WALL

Scheme of the kinematic mechanism

Table 6.9 Variables
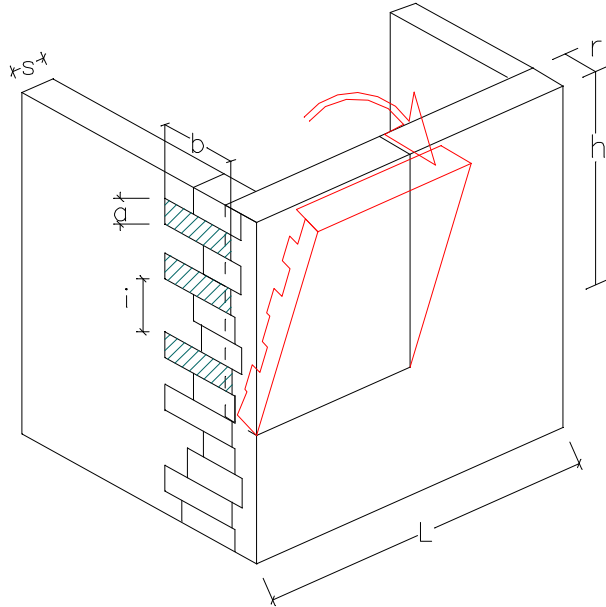
	Variables
r	Average thickness of the wall
h	Height of the wall
L	Free lenght of the façade (l = L / 2)
s	Average thickness of the orthogonal wall
i	Wheelbase of the connection blocks
b	Base of the connection blocks
a	Height of the connection blocks
φ	Friction angle

If the wall texture is able to connect continuously the façade to the orthogonal walls, a friction force that linearly increase downward opposed to the overturning is generated. If the façade is continuously connected through the wall texture to the perpendicular walls this force is:

$$F_a = -\gamma \cdot g \cdot s \frac{b}{2} tg\varphi (a + 2a + ... + h) \approx -\gamma \cdot g \cdot s \frac{b}{2} tg\varphi \frac{h^2}{2a} = -\frac{1}{2}\gamma \cdot g \cdot \frac{s \cdot h^2}{\rho} tg\varphi$$

where
$$\rho = \frac{i}{b} = \frac{2a}{b}$$

$$\gamma g \cdot (l + s)rh \cdot \frac{r}{2} + \frac{1}{2}\gamma g \cdot \frac{s \cdot h^2}{\rho} tg\varphi \cdot \frac{h}{3} = c \cdot \gamma g \cdot (l + s)rh \cdot \frac{h}{2}$$

$$c = \frac{r}{h} + \frac{1}{3}\frac{h}{r}\frac{s}{\rho(l + s)} tg\varphi$$

$$\frac{dc}{dh} = 0 \implies \frac{1}{3}\frac{s}{\rho(l + s)}\frac{tg\varphi}{r} - \frac{r}{h^2} \implies h = \sqrt{\frac{3\rho(l + s) \cdot r^2}{s \cdot tg\varphi}}$$

$$c_{min} = \sqrt{\frac{4 \cdot tg\varphi \cdot \frac{s}{l}}{3\rho\left(1 + \frac{s}{l}\right)}}$$

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OVERTURNING OF THE FAÇADE AND OF A WEDGE OF THE ORTHOGONAL WALL

Scheme of the kinematic mechanism

Table 6.10 Variables
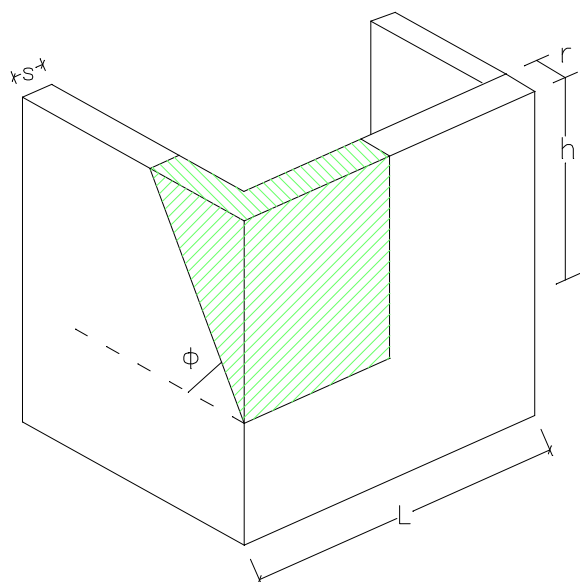
	Variables
r	Average thickness of the wall
h	Height of the
L	Lenght of the wall (l = L / 2)
b	Base of the connection blocks
a	Height of the connection blocks
φ	Friction angle
ϕ	Angle formed by the diagonal of the wedge

Stabilizing moment: $M_S = \gamma g \cdot rhl \cdot \dfrac{r}{2} + \gamma g \cdot \dfrac{h}{tg\phi} \dfrac{h}{2} s \cdot \dfrac{1}{3} \dfrac{h}{tg\phi}$

Overturning moment: $M_R = c\left(\gamma g \cdot rhl \cdot \dfrac{h}{2} + \gamma g \cdot \dfrac{h^2}{tg\phi} \dfrac{s}{2} \cdot \dfrac{2}{3} h \right)$

$c = \dfrac{h^2 s + 3 \cdot tg^2\phi \cdot r^2 \cdot l}{h \cdot tg\phi \cdot \left(2hs + 3rl \cdot tg\phi\right)}$

$\phi = arctg\sqrt{\dfrac{m}{tg\varphi}}$ where $m = \dfrac{2a}{b}$

$\dfrac{dc}{dh} = 0 \implies h = 2r \cdot tg\phi \cdot \left(1 + \sqrt{1 + \dfrac{3}{4}\dfrac{l}{s}}\right)$

$c_{min} = \dfrac{\sqrt{\dfrac{tg\varphi}{m}}}{1 + \sqrt{1 + \dfrac{3l}{4s}}}$

COLLAPSE OF A TRIANGULAR PORTION OF THE FAÇADE

Scheme of the kinematic mechanism   Table 6.11 Variables
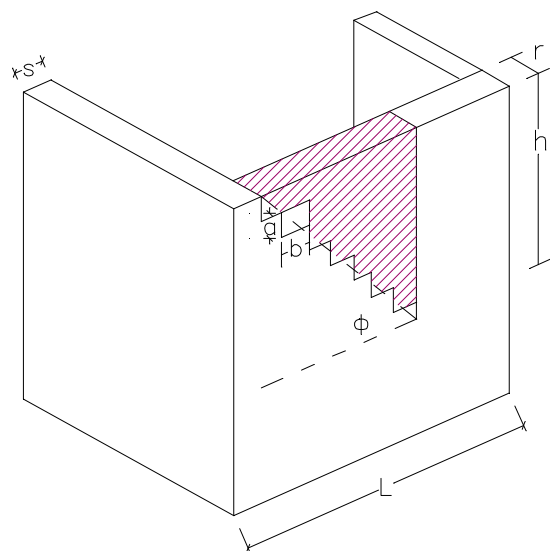
Variables	
r	Average thickness of the wall
L	Lenght of the wall
b	Base of the connection blocks
a	Height of the connection blocks
φ	Friction angle
ϕ	Angle formed by the detached portion

Giuffré: $$c = \frac{2r}{l \cdot m}$$

Rondelet: $$c = \frac{3}{4}\frac{r}{l \cdot m}\frac{1}{2}tg\,\varphi$$

where: $$m = 2\frac{a}{b}$$

$$a = \frac{b}{2}tg\phi \;\Rightarrow\; \phi = arctg\; m$$

$$h = l \cdot tg\phi \;\Rightarrow\; h = l \cdot m$$

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6.2.1.2 Mechanisms associated to horizontal stripes of masonry (last floor):

BENDING LIMIT FOR THE EMBEEDED BEAM SCHEMATIZATION (Mechanism 2.3.1 - Annex 1
Deliverable 3.1)

Scheme of the kinematic mechanism
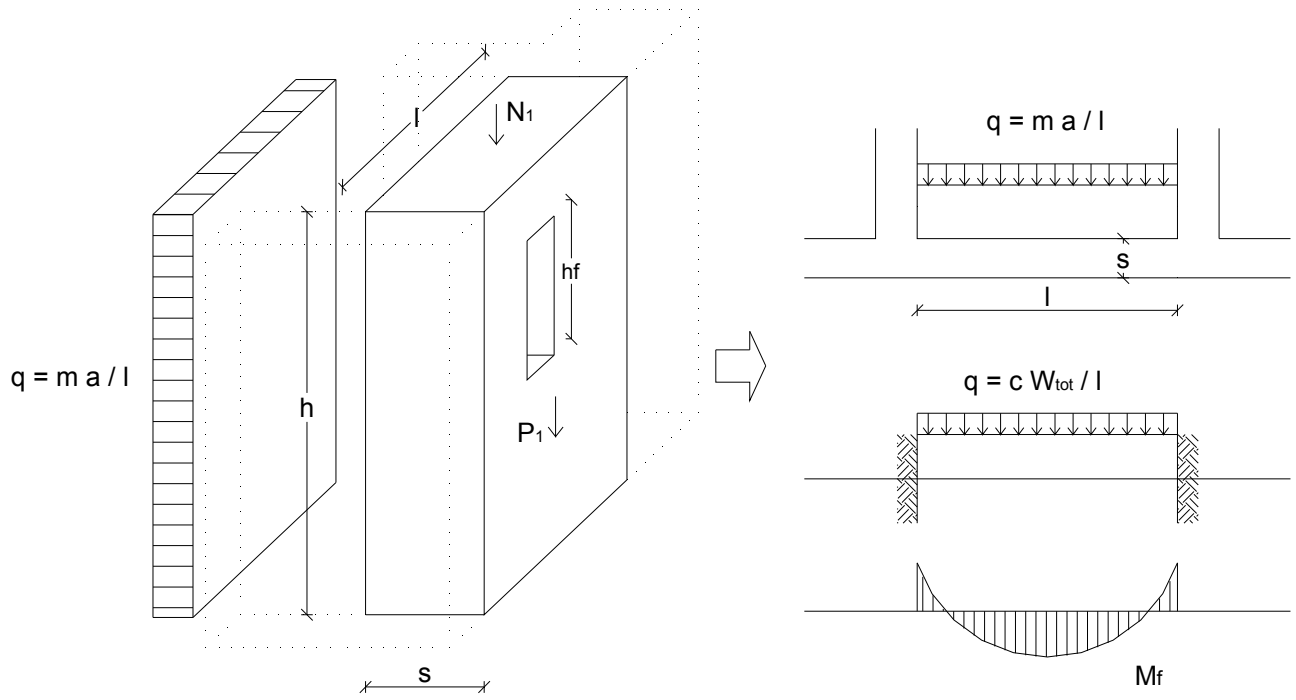
$$\sigma_t = \frac{M}{W_{res}} = \frac{\dfrac{1}{12} \cdot \dfrac{W_{tot}}{g} \cdot a \cdot \dfrac{l^2}{l}}{\dfrac{(h - h_f) \cdot s^2}{6}} \quad \rightarrow \quad c = 2 \cdot \sigma_t \frac{(h - h_f) \cdot s^2}{W_{tot} \cdot l}$$

where $W_{tot} = P_1 + N_1$

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COMPRESSION LIMIT FOR THE IN-THICKNESS-ARCH SCHEMATIZATION (Mechanism 2.3.2 - Annex 1 Deliverable 3.1)

The mechanism considers a resistant arc with a thickness of 1/5 of the thickness s of the wall and an arrow f = 4/5·s'. The arch is loaded with a uniformly distributed load q:

$$H = \frac{q \cdot l^2}{8 \cdot f} = \frac{q \cdot l^2}{8 \cdot (0.8 \cdot s)} = \frac{q \cdot l^2}{6.4 \cdot s}$$

Scheme of the kinematic mechanism
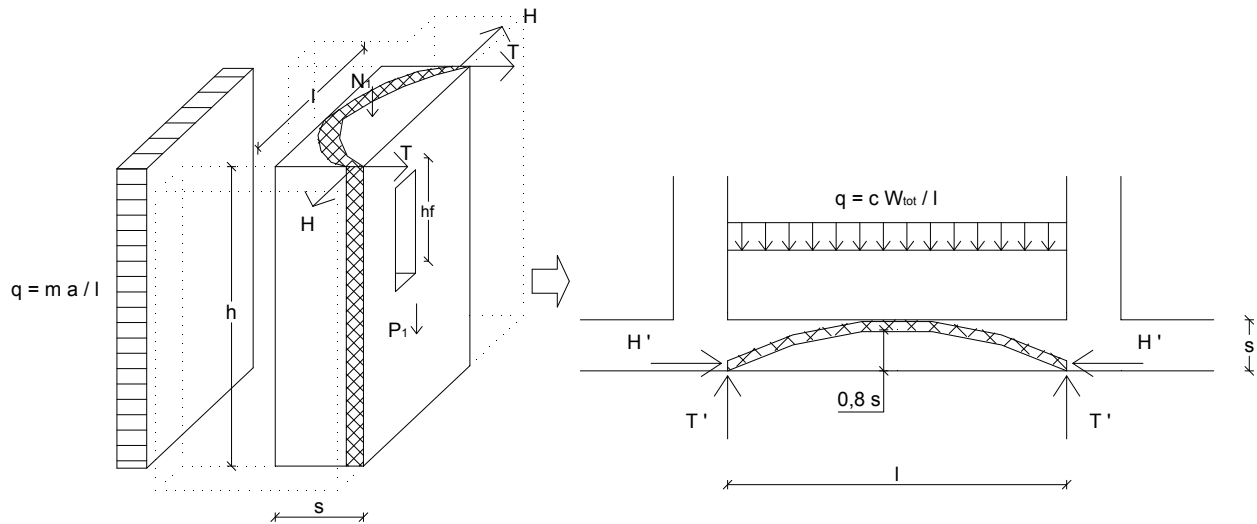
$$\sigma_c = \frac{N}{A} = \frac{\frac{1}{6.4 \cdot s} \cdot \frac{W_{tot}}{g} \cdot a \cdot \frac{l^2}{l}}{0.2 \cdot s \cdot (h - h_f)} \quad \rightarrow \quad c = 1.28 \cdot \sigma_c \frac{(h - h_f) \cdot s^2}{W_{tot} \cdot l}$$

where $W_{tot} = P_1 + N_1$

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DETACHMENT FROM THE TRANSVERSAL WALL (Mechanism 2.3.6 - Annex 1 Deliverable 3.1)

Scheme of the kinematic mechanism
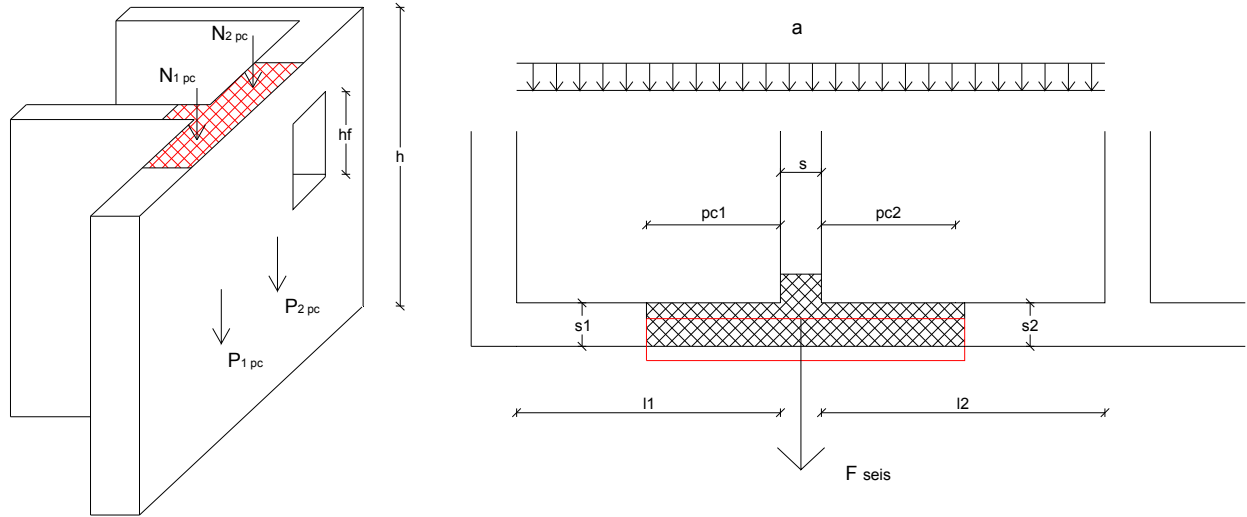
$$F_{seism} = \frac{a}{g}\left(W_1 + W_2\right)$$

$$F_{res} = \sigma_t \cdot s \cdot \left(h - h_f\right)$$

$$c = \frac{\sigma_t \cdot s \cdot \left(h - h_f\right)}{W_1 + W_2}$$

where $W_1 = P_1^{pc} + N_1^{pc}$, $W_2 = P_2^{pc} + N_2^{pc}$

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6.2.2 In plane mechanisms

MULTIPLE WALL SYSTEM (Mechanism 2.1.2 - Annex 1 Deliverable 3.1)

Scheme of the kinematic mechanism

Table 6.12 Variables
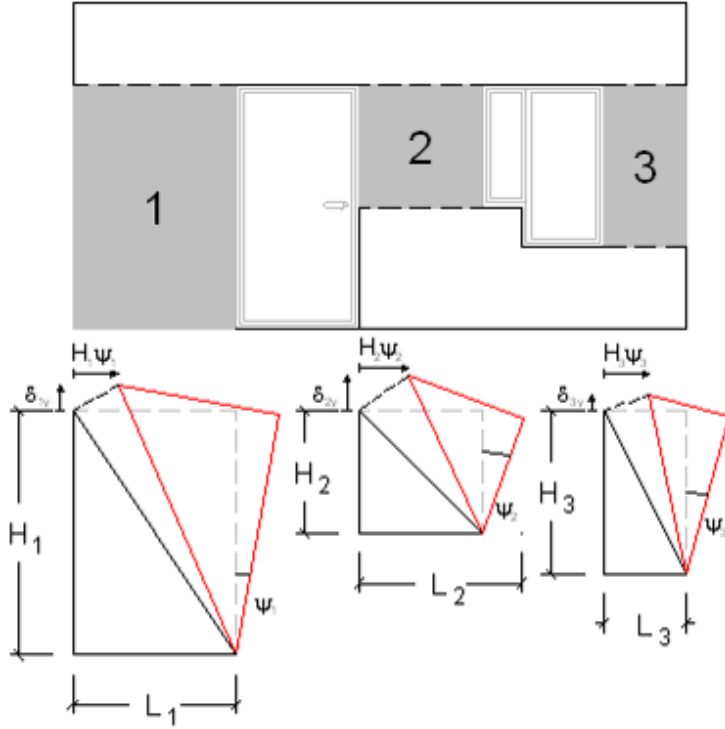
	Variables
H_1	Height of the wall 1
L_1	Lenght of the wall 1
N_1	Load acting on the wall 1
H_2	Height of the wall 2
L_2	Lenght of the wall 2
N_2	Load acting on the wall 2
H_3	Height of the wall 3
L_3	Lenght of the wall 3
N_3	Load acting on the wall 3
s	Average thickness of the wall

The value of the α coefficient that defines the position of the vertical load should be assumed equal to 1, if it is noted that, in the rotation of the panel, the contact with the upper portion of the wall is concentrated in one point. Actually local deformations that move the contact point to the compressed edge were observed. Conventionally, $\alpha = 0.75$ can be assumed. N is therefore the load acting on top of the panel at a distance αL from the compressed edge.

Principle of virtual work:

$$-P_1 \delta_{P_{1_y}} - P_2 \delta_{P_{2_y}} - P_3 \delta_{P_{3_y}} - N_1 \delta_{N_{1_y}} - N_2 \delta_{N_{2_y}} - N_3 \delta_{N_{3_y}} +$$
$$+ c \left(P_1 \delta_{P_{1_x}} + P_2 \delta_{P_{2_x}} + P_3 \delta_{P_{3_x}} + N_1 \delta_{N_{1_x}} + N_2 \delta_{N_{2_x}} + N_3 \delta_{N_{3_x}} \right) = 0$$

$$c = \frac{P_1 \dfrac{L_1}{3} + P_2 \dfrac{L_2}{3} \dfrac{H_1}{H_2} + P_3 \dfrac{L_3}{3} \dfrac{H_1}{H_3} + N_1 \alpha L_1 + N_2 \alpha L_2 \dfrac{H_1}{H_2} + N_3 \alpha L_3 \dfrac{H_1}{H_3}}{P_1 \dfrac{2}{3} H_1 + P_2 \dfrac{2}{3} H_1 + P_3 \dfrac{2}{3} H_1 + N_1 H_1 + N_2 H_1 + N_3 H_1}$$

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6.2.3 The FaMIVE procedure, (D'Ayala 2005)

The seismic performance of masonry buildings is directly related to the mechanical quality of the masonry fabric: a layout of regular stones of similar size and well staggered from one course to the next, with sufficient through thickness connections, will clearly ensure a better structural integrity. The comparison of failure mechanisms associated with rubble, as opposed to regular dressed stone masonry, eloquently illustrate this point.

Experimental work carried out in the past decades, (Tomaževič and Anicic 1989; Ceradini 1992; Giuffrè 1993), has shown that both the masonry fabric and the level of connection among orthogonal walls play an essential role in the seismic response of these buildings. Given the observed correlation, it is feasible to identify recurring structural details which characterize the seismic behavior and to correlate the presence of these with specific capacity, damage patterns and collapse mechanisms.

Following an approach first proposed by Giuffrè (1991) the FaMIVE procedure, (Failure Mechanism Identification and Vulnerability Evaluation), (D'Ayala and Speranza, 2002), models the masonry fabric as an ideal opus quadratum, characterized by a perfectly regular layout of the units, leading to the highest possible level of contact among them, and hence integrity of the whole. The results obtained, (D'Ayala and Speranza 2003), are in good agreement with experiments aimed at assessing the collapse load factor for in plane failure of blockwork walls without bonding agents, (Ceradini, 1992). The two assumptions of the model, relating to perfect regular staggering and to the contact surface perfectly coinciding with the geometric surface are clearly abstractions from reality, especially in cases in which the masonry units are only roughly squared and of variable size. The FaMIVE procedure takes this into account by a proportional reduction of the staggering ratio, as a function of the irregularity of the units. The calibration of these coefficients has been carried out through direct comparison of different masonry fabrics and of results of experimental work published by Baggio and Carocci (2000).

The programme FaMIVE, using the concepts of limit state analysis for non-conforming materials, correlates collapse mechanisms to specific constructional features of the external bearing walls forming a masonry building, (D'Ayala et al. 1997; D'Ayala 1999a). The analysis is static equivalent and quantifies the collapse load factor, (as a percentage of gravity acceleration - g), associated with each mechanism so as to determine a lower bound of the level of shaking which will trigger the onset of a specific failure mechanism. On this basis, it is possible to produce a prediction of most probable damage modes and levels of vulnerability for individual or groups of buildings, in relation to expected levels of shaking at a site. It is also possible to analyse the reduction in vulnerability obtainable by introducing selected types of strengthening.
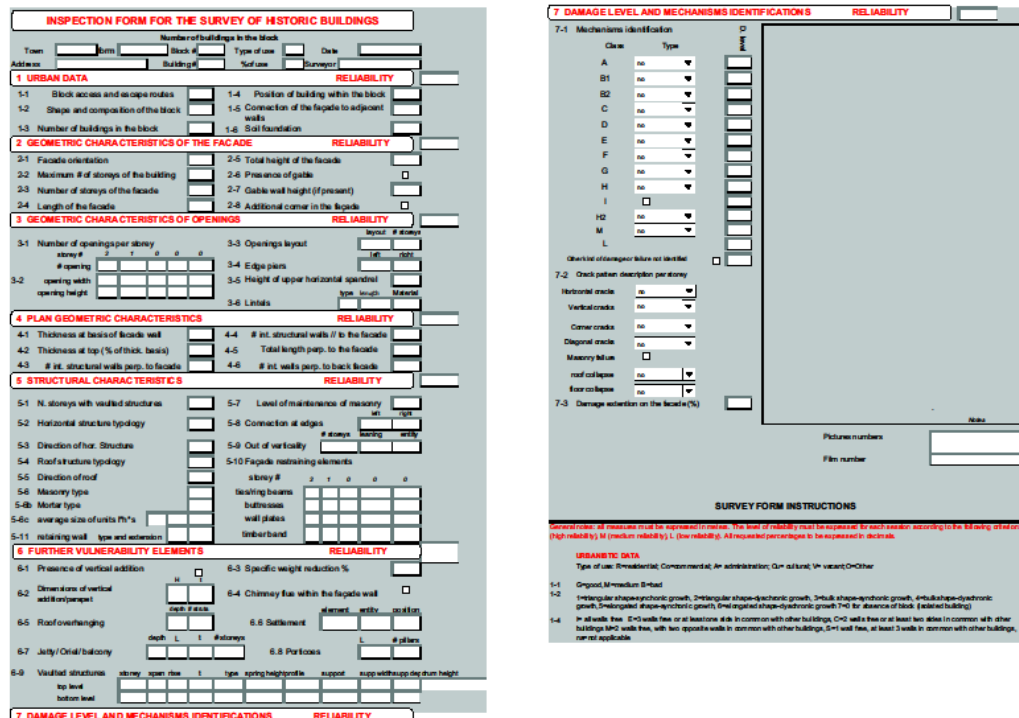
The methodology can be applied to medium size samples of buildings without forfeiting a sufficiently detailed analysis of the geometric, typological and structural parameters that characterised the analysed buildings.

This specific feature of FaMIVE is strictly related to the way in which the data collection is organised: the on-site inspection concentrates on those parameters which can directly influence the seismic performance of masonry buildings and can be satisfactorily surveyed from the street. In this way, the method minimises the surveying time and the need for pre-existing information (plans, etc...) while providing an analytically based vulnerability assessment. During the survey, the operator is required to identify, within the urban centre under study, recurring typological layouts, masonry fabrics, quality of materials and workmanship, expressing the level of reliability of the observation. Each of the identified typologies are further analysed by a detailed survey of a few specimens. This set of data directly relates to the local construction techniques and availability of materials, as usually only a limited number of architectural, structural and material typologies are present in a given urban centre. Once these are classified, for each building façade, the surveying activity from the street consists in recognising the pertinence to a given class for a specific feature, (masonry type, floor type, connection, etc…), and then the associated set of data is directly applied.

In particular, the FaMIVE survey form, (Figure 6.6), includes the description of:

- Urban data, (position of building within a block and connection to adjacent buildings);

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- The geometric characteristics of the façade, (orientation, dimensions, number of storeys);
- The geometric characteristics of openings, (lay-out, piers, lintels);
- The geometry in plan, (walls perpendicular to the façade);
- Structural characteristics, (type of horizontal structures, presence and lay-out of reinforcement, type and quality of masonry);
- Presence of further element of vulnerability, (additions, balconies, vaults);
- Typology and level of damage.
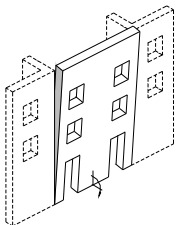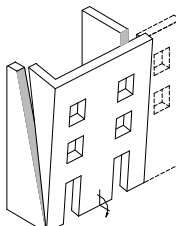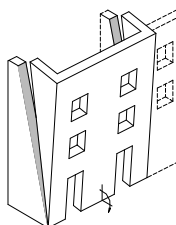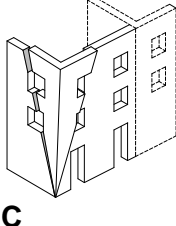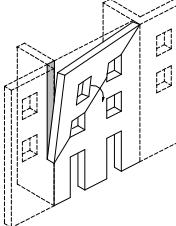
Figure 6.6 FaMIVE survey form.
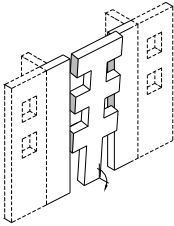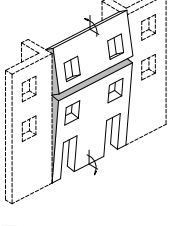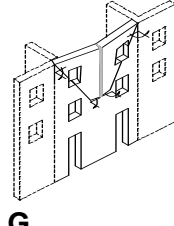
FaMIVE is implemented as a number of macros directly linked to an electronic database filled in on-site by use of an electronic form. It calculates the seismic vulnerability of each façade as a function of the collapse load factor, the type of mechanisms and the extent of structure collapsing, according to the following formula:

$$V = \max\left(\frac{d_c d_e}{ESC}\right)_i \qquad \text{Eq. (6.1)}$$

where ESC, the collapse load factor (see Table 6.13 and Figure 6.8), is a function of the slenderness, the connection with other walls and floor structures, the masonry fabric and the friction coefficient; d_e and d_c are two coefficients measuring the extension of the façade and floor structures involved in the collapse and the catastrophic character of the collapse, respectively. The index i refers to the fact that for each façade a number of feasible mechanisms are considered, depending on constraint conditions and connections with other walls and floors. The term in parenthesis in Eq. (6.1) is calculated for each of the feasible mechanisms and the one yielding the maximum product V is taken as the measure of that façade vulnerability.

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Table 6.13 Mechanisms and load factors for façade failures, (D'Ayala and Speranza 2003).

Mechanism	Load factor
A	$$\lambda_{(0),j} = \frac{\sum_{i=1}^{j}\frac{T_i^2}{2}L + \beta\frac{h_s}{3}j^2\mu sb\sum_{l=1}^{r}\frac{l}{r} + kL\left[\frac{T_N}{2} + \mu h_s j + \sum_{i=1}^{j}\cdot\left(T_i + \Delta T + \mu h_s(j-i)\right)\right]}{h_s\left[\sum_{i=1}^{j}LT_i\left(j-i+\frac{1}{2}\right) + kL\left(j + \sum_{i=1}^{j}(j-i)\right)\right]}$$ β is the number of internal bearing walls orthogonal to the façade, and effectively connected to it as to provide restraining action due to friction
B1	$$\lambda_{(0),j} = \frac{\sum_{i=1}^{j}\frac{T_i^2}{2}L + (\varepsilon+\beta)\frac{h_s}{2}j^2\tan\alpha_j T_s\left(\frac{h_s}{3}\tan\alpha_j + T_i\right) + kL\left[\frac{T_N}{2} + \mu h_s j + \sum_{i=1}^{j}\cdot\left(T_i + \Delta T + \mu h_s(j-i)\right)\right]}{h_s\left[\sum_{i=1}^{j}LT_i\left(j-i+\frac{1}{2}\right) + (\varepsilon+\beta)\frac{h_s^2}{3}j^3\tan\alpha_j + kL\left(j + \sum_{i=1}^{j}(j-i)\right)\right]}$$ ε is the number of edge party walls orthogonal to the façade under exam which can provide restraining action $\varepsilon = 1$ β as above T_s thickness of party walls and internal bearing walls
B2	$$\lambda_{(0),j} = \frac{\sum_{i=1}^{j}\frac{T_i^2}{2}L + (\varepsilon+\beta)\frac{h_s}{2}j^2\tan\alpha_j T_s\left(\frac{h_s}{3}\tan\alpha_j + T_i\right) + kL\left[\frac{T_N}{2} + \mu h_s j + \sum_{i=1}^{j}\cdot\left(T_i + \Delta T + \mu h_s(j-i)\right)\right]}{h_s\left[\sum_{i=1}^{j}LT_i\left(j-i+\frac{1}{2}\right) + (\varepsilon+\beta)\frac{h_s^2}{3}j^3\tan\alpha_j + kL\left(j + \sum_{i=1}^{j}(j-i)\right)\right]}$$ β as above ε as above $\varepsilon = 2$
C	$$\lambda_{(0),j} = \frac{\theta_{jf}\left[\frac{T_N+T_j}{2}\frac{j^2}{2} + vk\sum_{i=0}^{j-1}(j-i)\right]\sqrt{\left(\frac{jh_h}{3}\theta_{jf}\right)^2 + \left(\frac{T_j}{2}\right)^2} + \theta_{js}\left[\frac{T_N+T_j}{2}\frac{j^2}{2} + vk\sum_{i=0}^{j-1}(j-i)\right]\sqrt{\left(\frac{jh_h}{3}\theta_{js}\right)^2 + \left(\frac{T_j}{2}\right)^2}}{\frac{2}{3}j\left[j^2\frac{T_N+T_j}{2}\frac{h_s}{2}\left(\theta_{jf}+\theta_{js}\right) + k\left(v\theta_{jf}+(1-v)\theta_{js}\right)\sum_{i=0}^{j-1}(j-i)\right]}$$ $\theta_{jf} = \tan\alpha_{jf}$ $\theta_{js} = \tan\alpha_{js}$ v integer identifies the load bearing wall $v = (0,1)$
D	$$\lambda_{(0),j} = \frac{\frac{T_{mj}}{2}\left[\frac{T_{mj}}{2}Lj + kL + kh_s\tan\alpha_j\sum_{i=1}^{j-1}(j-i)\right] + \left(1 + \sum_{\beta}\left(\frac{L_\beta}{L}\right)^2\right)\frac{h_s}{3}j^2\mu sb\sum_{l=1}^{r}\frac{l}{r} + \mu jkh_s\left[\frac{L}{2} + \frac{2}{3}h_s\tan\alpha_j\sum_{i=1}^{j-1}(j-i)\right]}{j\left[j\frac{LT_{mj}}{3} + kL\frac{h_s}{2} + \frac{2}{3}kh_s^2\tan\alpha_j\sum_{i=1}^{j-1}(j-i)\right]}$$ β as above L_β width of facade in between party walls. T_{mj} average thickness of wall over height of overturning portion

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E	$$\lambda_{(0),j} = \frac{\sum_{i=1}^{j}\frac{T_i^2}{2}L_{var} + \left[(\nu+\eta)(h_s - h_{op}) + \beta h_s\right]\frac{1}{3}j^2\mu sb\sum_{l=1}^{r}\frac{l}{r} + kL_{var}\left[\frac{T_N}{2} + \mu h_s j + \sum_{i=1}^{j}\cdot(T_i + \Delta T + \mu h_s(j-i))\right]}{h_s L_{var}\left[\sum_{i=1}^{j}T_i\left(j-i+\frac{1}{2}\right) + k\left(j+\sum_{i=1}^{j}(j-i)\right)\right]}$$ β as above ε as above $\varepsilon = (1,2)$ η is the number of vertical discontinuities within the façade non coincident with the façade edges $\eta = (1,2)$ $\nu = \varepsilon - \eta$, integer $\nu = (0,1)$ provides the number of active side connections
F	$$\lambda_{(h)} = \frac{\frac{T_i}{4}(T_i + T_k + k) + \frac{T_i\nu}{h_v}\left(\frac{T_N + T_i}{2}h_u + kh_s\right) + \frac{2sbh_v\mu(\nu-1)}{3l\nu^2}\left[\sum_{l=r_0}^{r_1}\frac{l}{r_1-r_0} + (\nu-1)\sum_{l=r_1}^{r_2}\frac{l}{r_2-r_1}\right]}{\frac{h_v(\nu-1)}{4\nu^2}\left[T_i - T_j + \nu(T_k + T_j)\right] + \frac{k(\nu-1)}{\nu}\left[h_v + h_i - h_j\right]} +$$ $$+ \frac{\frac{\sigma_u}{h_v 6\gamma}\left[T_i^2 + \nu T_k^2 + (\nu-1)T_j^2\right]}{\frac{h_v(\nu-1)}{4\nu^2}\left[T_i - T_j + \nu(T_k + T_j)\right] + \frac{k(\nu-1)}{\nu}\left[h_v + h_i - h_j\right]}$$ r_0, r_1, r_2, number of courses above upper hinge, middle hinge, lower hinge respectively $1/\nu$ real number $(0,1)$ h_v height of portion of wall subjected to mechanism
G	$$\lambda_{\alpha,hi} = \frac{\mu\left[\left(l_\alpha\frac{T_u + T_i}{4} + \frac{\cdot kL}{2i}\right)\frac{l_\alpha}{2} + \frac{(T_u + T_i)^2(L - l_\alpha)}{4} + sb(T_u + T_i)\sum_{l=1}^{r}\frac{l}{r} + s_\beta b_\beta l_\alpha\sum_{l_\beta=1}^{r_\beta}\frac{l_\beta}{r_\beta}\right]}{\left[\frac{(l_\alpha + L)(T_u + T_i)}{8} + \frac{kL}{2i}\right]\frac{5l_\alpha^2 + 5Ll_\alpha - L^2}{3(l_\alpha + l)}}$$ $l_\alpha = L - h_i 2tg\alpha_i \cdot$ valid for $2l_\alpha(l_\alpha + L) > L^2$ The index β identifies quantities associated with internal bearing walls

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Mechanism A

Mechanisms B1 and B2

Mechanism C

Mechanism D

Mechanism E

(a)   (b)   (c)

Mechanism E: portions of faÁade involved in the collapse in relation to the pier width
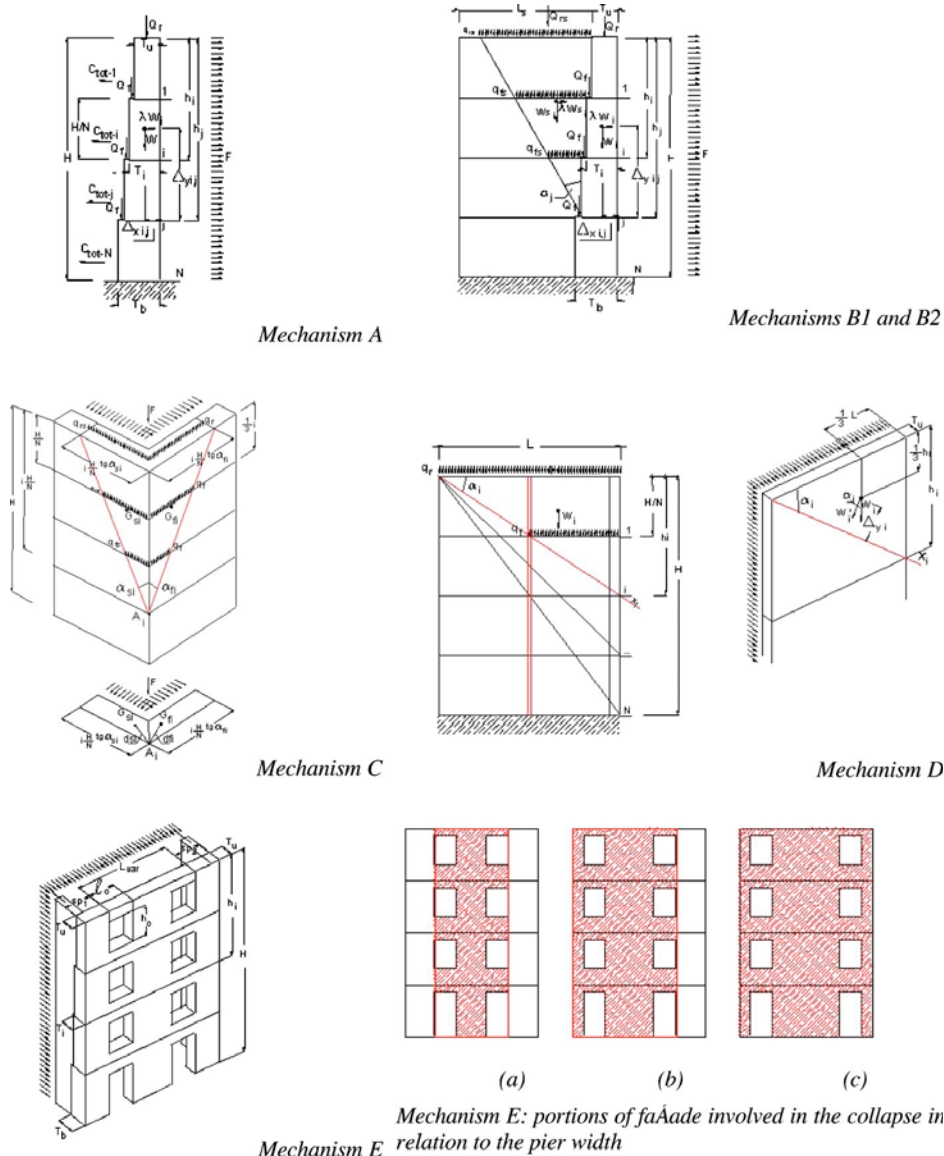
Figure 6.7 Geometric parameters and applied load for overturning mechanisms.

6.3   STRUCTURAL ELEMENT MODELS

Structural element models (SEM) approximate the actual structural geometry more accurately by describing individual structural elements such as column, piers, arches, vaults and walls with the assumption of homogeneous material behavior.

It is possible to model a masonry structure as a set of two-dimensional masonry panels or one-dimensional elements. In the second case, the building is subdivided in structural elements (masonry walls and bands) and it is transformed in short beams with non-linear behavior or trusses and struts, (Magenes et al. 2000).

Each model (one or two-dimensional) has no tensile strength and for two-dimensional modelling the element stiffness changes with the compression strength.

In literature there are two different techniques to implement the no tensile strength assumption:

- change the geometry of structural elements to remove the tensile strain area, (D'Asdia et al. 1994);
- modify the strength inside the two-dimensional macro element, (Braga et al. 1990).

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Both models use linear elastic behavior, as so it will be necessary control the maximum compression stress. For the one-dimensional elements only the effective area of the masonry is modelled with a strut where the orientation and stiffness is considered equal to the one of the masonry walls. The model fails when the limit equilibrium configuration is achieved or the maximum compression strength is verified on the strut.

Another set of one-dimensional models use beams elements with shear deformation. These models are subdivided in changing stiffness elements, (Braga et al. 1982), and constant stiffness elements in elastic behavior followed by plastic deformation, (Tomaževic 1978; Dolce 1989, Tomaževic et al. 1990). In this last case, the nonlinearity of the behavior is triggered by reaching a limit resistance condition. Most of the methods based on the "In-plane mechanism" (like for example POR) fall into this class of models.

Another important element of distinction between the methods is the number of possible inelastic behavior mechanisms, and especially the failure mechanisms of the individual structural elements and of the all structure. On the POR method the main limitations in its original version (Tomaževic 1978; DT2 1978), consisted on:

 a) considering the masonry walls as the only deformation and collapse elements, without assessing the possible failure of the interaction strut elements;

 b) on hypothesizing a single possible failure mechanism of masonry walls (shear failure with diagonal cracking), neglecting the out-of-plane and sliding failure.

Subsequent proposals for improvement of the method, (Dolce 1989, Tomaževic and Weiss 1990), have solved the problem b) in a fairly easily way by introducing additional appropriate failure criteria. However it was not possible to satisfactorily address the limitation a), as a consequence of another fundamental limit of the method regarding the assembled structural model.

In fact, the model is based on the assumption of "in-plane mechanism" and performs a shear-displacement nonlinear analysis separately for each floor. This approach, which greatly simplifies the calculations, may not take into account the problem of calculating solicitation on the interaction strut elements if not by using very approximate calculations, (Braga and Dolce 1982; Fusier and Vignoli 1993). Among other things, a shear-displacement inter-storey analysis requires that assumptions are made on the existing degree of restraint at the masonry walls boundaries (extremities). This degree of constraint depends on the stiffness and strength of the horizontal coupling elements (interaction strut elements and / or R.C. beams), which are increasingly solicited with the increase of the horizontal seismic forces, and as so are susceptible to cracking or collapse. It is obvious that these phenomena can be evaluated in a sufficiently accurate way by performing a global analysis of multi-storey wall or of the entire building.

The overall SEM analysis of the building is also the only way to prevent violations of local and global equilibrium: it has been highlighted that a floor by floor separate analysis cannot account for the variations in the axial force on the masonry walls due to the increase of the seismic forces, that may affect the stiffness but above all the resistance of these structural elements.

Finally it is presented the method proposed by the University of Genova researchers, (Gambarotta and Lagomarsino 1996; Brencich and Lagomarsino 1997 and 1998a). Important research efforts have been devoted to the development computational approaches based on rigid and deformable macro-elements. Each macro-element models an entire wall or masonry panel, reducing drastically the number of degrees of freedom of the structure. Brencich, Gambarotta and Lagomarsino (1998, Figure 6.8) use two nodes macro-elements, take into account the overturning, damage and frictional shear mechanisms experimentally observed in masonry panels. The overall response of buildings to horizontal forces superimposed to the vertical loads is obtained assembling shear walls and flexible floor diaphragms. The former are made up of both macro-elements, representative of piers and spandrels, and rigid elements, representing the undamaged parts of the walls.

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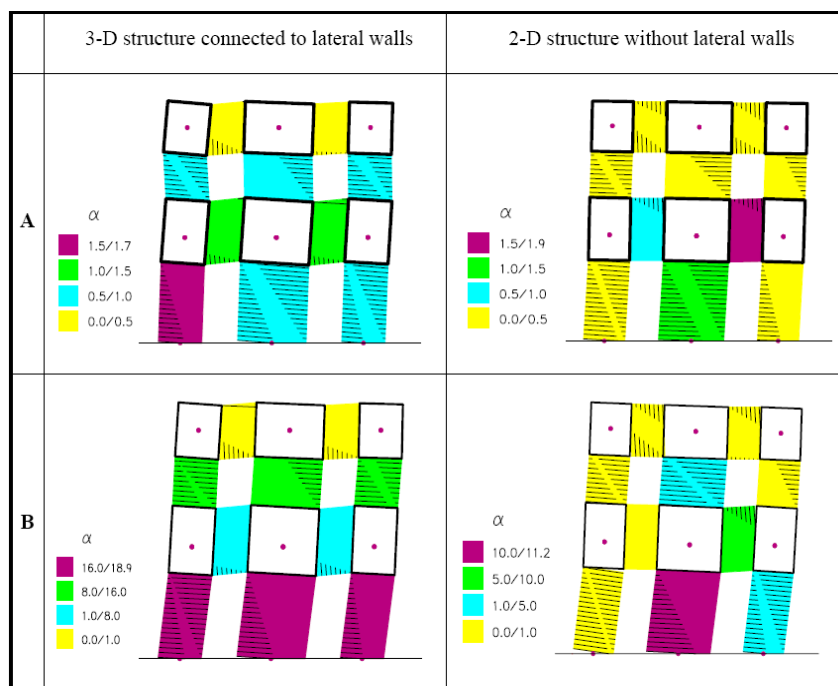
Figure 6.8 Analysis with macro-blocks of a façade wall. Damage distribution under monotonic loading, (Brencich 1998b), at peak load (A) and at the end of the monotonic load history (B).

This model differs from the previously presented models because its main purpose is to be used on the modelling of the cyclic behavior of masonry walls, although it can be used for nonlinear static analysis. As this is a macro-element model, it has therefore the great advantage of allowing dynamic analysis with a relatively small computational cost. According to the criteria previously discussed, the classification of the model it is not immediate.

In fact, the kinematic and static parameters used in the formulation of the element consist on nodal displacements and rotations and resulting forces Moment, Shear, Normal strength, which recall the one dimensional models.

However, the introduction of internal degrees of freedom and of appropriate considerations on the out-of-plane or rocking and shear-sliding mechanisms confer a bi-dimensional character to the element, which seems to simulate in an effective and concise way the most important characteristics of the nonlinear response of the wall panels. Nowadays, the limit of this method is the need for a posterior calibration of the constitutive law parameters in order to obtain comparable results with experiments or with more refined methods of analysis. Nevertheless, the ability to reproduce the cyclic response (and thus the phenomena of hysteresis energy dissipation associated with it) makes the method a useful and versatile tool both on the research and on practical applications.

More recent developments, as those due to Casolo and Peña (2007) and Chen and Moon (2008), show the permanent interest of this type of simplified approaches which and their ability to combine satisfactory accuracy with computer efficiency.

Casolo and Peña (2007) have developed a specific rigid element approach for the in-plane dynamic analysis of masonry walls. A rigid body spring model (RBSM) has been adopted consisting of a collection of plane quadrilateral rigid elements connected to each other by two normal springs and one shear spring at each side. Specific separate hysteretic laws are assigned to the axial and shear deformation between elements. A Coulomb-like law is adopted to relate he strength of the shear springs to the vertical axial loading. The satisfactory performance of the approach has been proven by comparison with available experimental and numerical results on pier walls and façades. The technique has been also successfully applied to the study of a large real masonry construction, namely the Maniace Castle in Syracuse, (Casolo and Sanjust 2009).

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Molins et al (1998) developed a method for the analysis of masonry skeletal structures as an extension of conventional matrix calculation to systems composed of curved members with variable cross-section. The method includes a set of partial models for the description of the non-linear response of masonry taking into account cracking in tension and yielding or crushing in compression. Roca et al. (2005) extended the method to analyse 3D systems including masonry load bearing walls using Kwan's proposal for the modelling of wall systems as equivalent frames. The method has been successfully used in the assessment of façades and entire buildings.

6.4 FINITE ELEMENT METHOD

6.4.1 Introduction

The finite element method is one of the possible and most used approaches for the modelling of structures. The finite element method offers a widespread variety of possibilities concerning the description of the masonry structures within the frame of detailed non-linear analysis. Most of modern possibilities based on FEM fall within two main approaches: modelling at the micro level, considering the material as discontinuous, or at the macro level. Hybrid models can also be created, these models have considerable interest when, for example, it is intended to analyze in detail a specific structural element within a more complex structure.

Linear elastic analysis is commonly used in the calculation of steel and reinforced concrete structures. However, its application to masonry structures is, in principle, inadequate because it does not take into account the non-tension response and other essential features of masonry behavior. It must be noted that, due to its very limited capacity in tension, masonry shows a complex non-linear response even at low or moderate stress levels. Moreover, simple linear elastic analysis cannot be used to simulate masonry strength responses, typically observed in arches and vault, characterized by the development of partialized sub-systems working in compression. Attempts to use linear elastic analysis to dimension arches may result in very conservative or inaccurate approaches. Linear elastic analysis is not useful, in particular, to estimate the ultimate response of masonry structures and should not be used to conclude on their strength and structural safety.

Notwithstanding, linear elastic analysis has been used, with partial success, as an auxiliary tool assisting in the diagnosis of large masonry structures. Easy availability and reduced computer costs have promoted its use, in spite of the mentioned limitations, before the development and popularization of more powerful computer applications.

Some examples are the studies of San Marco in Venice by Mola and Vitaliani (1995), the Metropolitan Cathedral of Mexico (Meli 1995), the Tower of Pisa (Macchi et al. 1993), the Colosseum of Rome (Croci 1995) and the Church of the Güell Colony in Barcelona, by Gonzalez et al. (1993), see also and Roca (1998), among many others. The case of Hagia Sophia deserved much attention and has been analyzed by different authors using similar modelling techniques (e.g. Mark et al 1993a, 1993b, Croci et al. 1997). In all these cases, the limitations of the method were counterbalanced by the very large expertise and deep insight of the analysts.

Meli and Peña (2004) have discussed the possibilities of elastic-linear models in providing preliminary information for the seismic study of masonry churches and the possibility of using information obtained from them for constructing more detailed models of critical parts to be then studied separately with more complex analyses.

Micro-modelling

Some authors, such as Silva in (Silva et al. 2008b) or Costa in (Costa et al. 2007) on the analysis of stone masonry structures through the finite element method, used a detailed micro-modelling approach, reducing the masonry to its basic components, (joints, blocks and infill), Figure 6.9. In this so-called detailed micro-models describe the units and the mortar at joints using continuum finite elements, whereas the unit-mortar interface is represented by discontinuous elements accounting for potential crack or slip planes (**Errore. L'origine riferimento non è stata trovata.**).

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 This type of detailed modelling at the micro level has a greater degree of accuracy; however, it is inevitably accompanied by an increase in the calculation time and effort, which makes this modelling strategy unfeasible in the current study of real structures. It is a type of modelling suitable for the study of localized areas where it is intended to study the local effects and where there is detailed knowledge of the geometry and composing elements. It is particularly adequate to describe the local response of the material. Elastic and inelastic properties of both unit and mortar can be realistically taken into account.

(a)

(b)
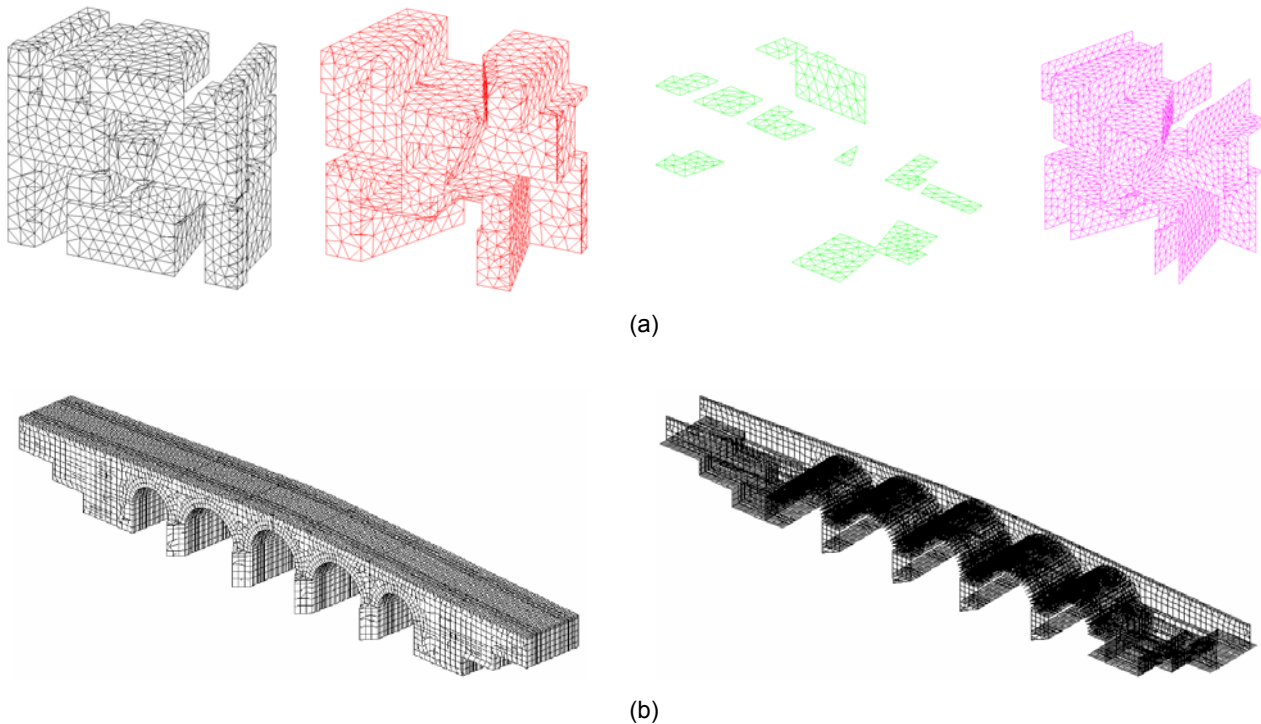
Figure 6.9 Micro-Modelling using Finite Elements. (a) Azores wall, (Silva 2008b). (b) Lagoncinha Bridge, (Costa 2007).

The detailed micro-modelling strategy leads to very accurate results, but requires an intensive computational effort. This drawback is partially overcome by the simplified micro-models. Some authors, (Lourenço 1996; Lotfi and Shing 1994; Tzamtzis 1994; Lourenço and Rots, 1997, (**Errore. L'origine riferimento non è stata trovata.**); Gambarotta and Lagomarsino 1997; Sutcliffe et al. 2001), opt for this simplified micro-modelling strategy, which is characterized by the combination or omission of certain constituents, allowing to drastically reduce the computation time without a great loss   of   accuracy.
In this type of modelling the expanded units, which are represented by continuum elements, are used to model both units and mortar material, while the behavior of the mortar joints and unit-mortar interfaces is lumped to the discontinuous elements, (**Errore. L'origine riferimento non è stata trovata.**). Masonry is thus considered as a set of elastic blocks bonded by potential fracture/slip lines at the joints.

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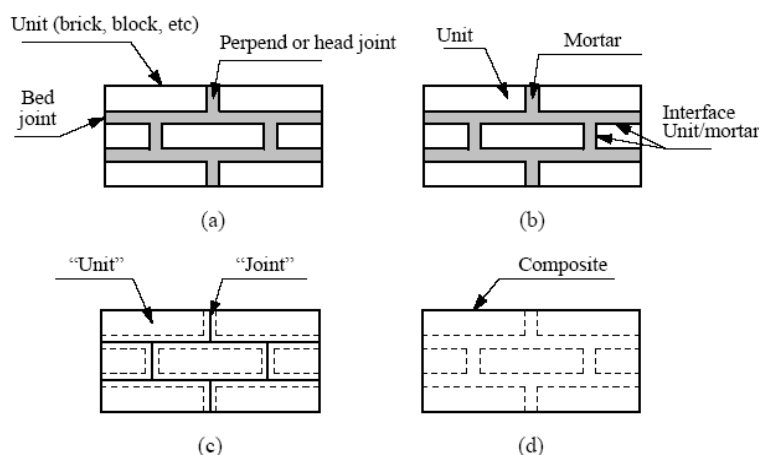
Figure 6.10 Modelling strategies for masonry structures, (Lourenço 1996): (a) masonry sample; (b) detailed; and (c) simplified micro-modelling; (d) macro-modelling.
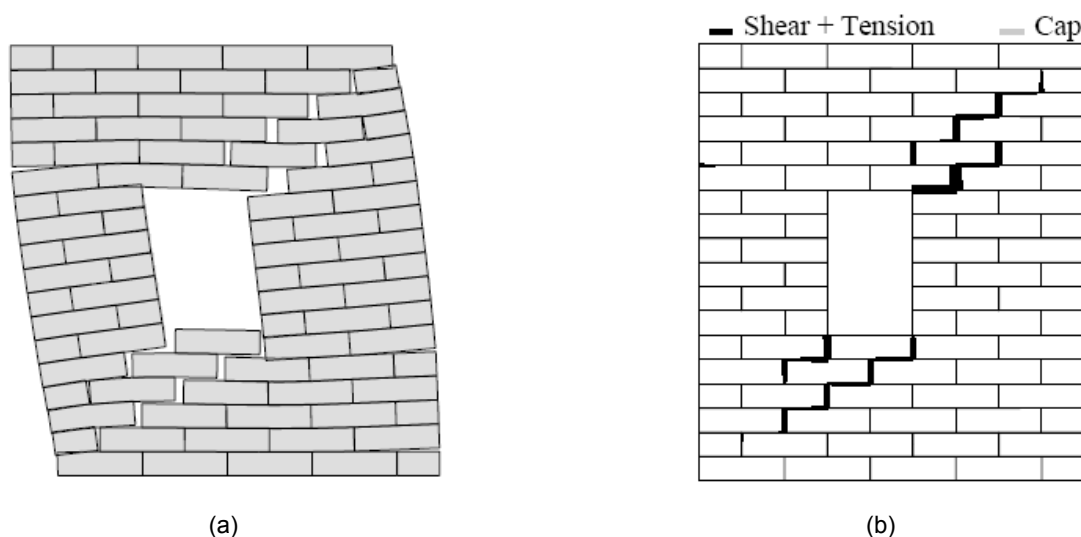
Figure 6.11 Micro-modelling of masonry shear walls (Lourenço 1996). Results of the analysis at a lateral displacement of 2.0 mm: (a) deformed mesh; (b) damage.

The micro-modelling approaches are suitable for small structural elements with particular interest in strongly heterogeneous states of stress and strain. The primary aim is to closely represent masonry based on the knowledge of the properties of each constituent and the interface. The necessary experimental data must be obtained from laboratory tests on the constituents and small masonry samples. Nevertheless, the high level of refinement required means an intensive computational effort, (i.e. great number of degrees of freedom of the numerical model), which limits micro-models applicability to the analysis of small elements, (e.g. laboratory specimens), or small structural details.

Macro-modelling

Also in the field of finite element modelling, some authors opt for the macro-modelling using macro mechanical models, also known as homogeneous or continuous, in which all elements of an assembly of materials are incorporated into a continuum, for which is established a relation between the average extensions and stresses of the masonry. These relations are obtained by adopting a phenomenological point of view or using homogenization techniques.

Macro-modelling is probably the most popular and common approach due to its lesser calculation demands. In practice-oriented analyses on large structural members or full structures, a detailed

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description of the interaction between units and mortar may not be necessary. In these cases, macro-modelling, which does not make any distinction between units and joints, may offer an adequate approach to the characterization of the structural response.

The macro-models have been extensively used with the aim of analyzing the seismic response of complex masonry structures, such as arch bridges (Pela' *et al.* 2009), historical buildings (Mallardo *et al.* 2008), and mosques and cathedrals (Roca *et al.*, 2004, Martínez *et al*, 2006; Murcia-Delso et al. 2009).

The smeared crack scalar damage models or other similar models, such as those presented in (Faria et al. 1994) and (Cervera 2003) are often used in macro-modelling of masonry. This type of models, where the damage (d) is defined in a given point by a scalar value which defines the level of material degradation, that ranges from the elastic state until collapse, and the cracking is considered as distributed along the structure, are commonly used in the modelling/analysis of reinforced concrete structures, (Faria et al. 2004), or large volumes of concrete, (Faria et al. 1994).

Despite several simulations performed with these models, show that the damage appears spread in a vast volume of the analyzed structures, unlike what happens in reality, the results are quite satisfactory, since the vulnerabilities of a structure can be easily detected, (Silva 2008a), Figure 6.12.
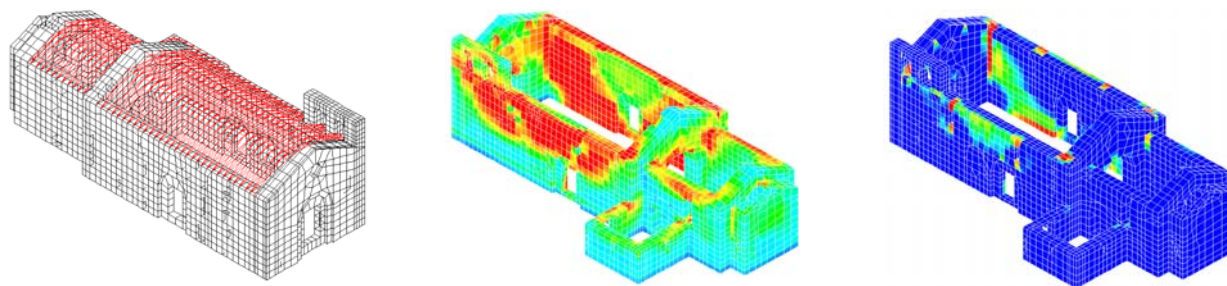
Figure 6.12 Macro-Modelling of the Gondar church using Finite Elements, (Silva 2008a).

Silva in (Silva et al. 2008a) concluded that this type of model is capable of capturing the main global phenomena of masonry walls. However, the local behavior, highly dependent on the geometrical particularities, can hardly be simulated with this model. In particular, the model cannot represent the sliding at joint level which reflects a highly nonlinear behavior that such a model does not include. These phenomena are all the more important when the dimension of the walls decreases in relation to the size of the stones and joints. In a large structure such as a church, in comparison with the structures tested in the laboratory, the local phenomena, characteristic of masonry, dilute themselves in the overall behavior, tending the material behavior to the behavior of a homogeneous material for which the damage model was created. As a result, "it is believed" that this model is able to better reproduce the actual behavior of a full structure than the behavior of the structural elements tested in laboratory.

The use of a continuum damage model presented thus a clear advantage over a simple linear analysis, since the consideration of the cumulative development of damage on the structure, both in traction and compression, provides a better understanding of how it spreads and of the maximum levels of effort that the structure is subject to.

In response to the fact that the damage appears distributed in an unrealistic way, several authors adopted this model as a starting point and applied changes, with some success, in order to quantify such characteristics; an example of that is presented in (Clemente et al. 2006).

In (Clemente et al. 2006) an interesting enhancement to smeared damage representation was proposed. The method is based on the so-called smeared-crack scalar damage model, modified in such a way that it can reproduce localized individual (discrete) cracks. This is achieved by means of a local crack-tracking algorithm. The crack tracking model enables the simulation of more realistic damage distributions than the original smeared-crack model. The localized cracks

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predicted by the crack tracking model behave, in consistency with limit analysis, as a set o hinges developing gradually and finally leading to a full collapsing mechanism. The model has been used to analyze the response of the structure of Mallorca Cathedral under gravity and seismic forces (**Errore. L'origine riferimento non è stata trovata.**). More recently, Clemente's et al. (2006) isotropic damage has been modified to account for masonry's orthotropy by Pela' et al. (2008, see also Pela' 2009).
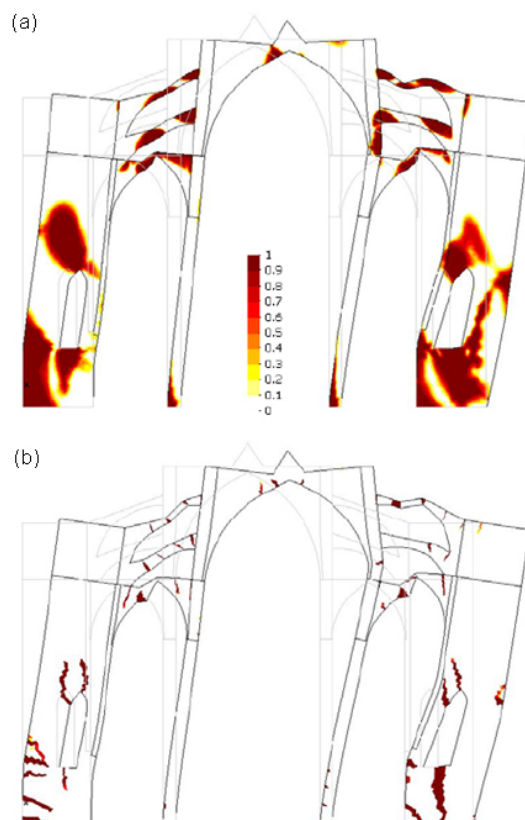
Figure 6.13 Seismic analysis of Mallorca Cathedral. Smeared damage approach (a) versus localized damage approach (b), for an earthquake of 475 years of return period (Clemente et al. 2006). Both diagrams represent the tensile damage scalar parameter in chromatic scale.

In (Saetta et al. 2000) it is proposed a model developed for the study of fragile masonry subjected to in-plane loading. It is a continuum damage model with four independent internal damage parameters in compression and tension and for both natural directions of masonry. The definition of these parameters allows the recovery of stiffness with the closing of cracks as well as the different inelastic behavior along each considered axis. For each of the damage parameters where defined evolution laws and a damage criterion, both expressed in terms of equivalent stress. In this model, the evolution laws of damage were assumed similar to those proposed by Faria in (Faria, 1994) for concrete.

In (Berto et al. 2001), this model has undergone some upgrades, to improve its ability to simulate the mechanical behavior of masonry. To the model were added, a shear factor, which allows taking into account the friction effect along a crack and the possibility of considering irreversible strains.

Lagomarsino presented in (Lagomarsino et al. 1997) a mortared joint model for the analysis of masonry. It is a model based on the damage mechanics that takes into account both the damage to the infill and the lack of cohesion between the masonry and infill, which occurs when opening and frictional sliding phenomena happens. The constitutive equation depends on two internal variables that represent the frictional slip and damage of the mortared joints, being these variables governed by a slip boundary and by a damage condition. This model is characterized by a frictional dissipation along with a degradation of stiffness under compressive stress.

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This model was used by these authors in two different strategies of modelling, on the first, presented in Part I, (Lagomarsino et al. 1997), this model was applied to the composed modelling of masonry walls where the blocks and the mortared joint were modeled individually (micro modelling). In the second part of this work, the mortared joint damage model served as basis for a continuum damage model, which was used on the modelling of large-scale walls. The constitutive equations for masonry were obtained based on a homogenization process that involved the mortared joint damage model proposed and simple constitutive damage equations for the block layer.

In (Calderini et al. 2008) it is presented a homogenized continuum model for the study of inelastic and anisotropic in-plane behavior of masonry. The constitutive equations of the homogenized continuum are obtained through micro mechanical analysis, having been defined for that a reference volumetric element. This type of analysis is valid for application in materials where it is possible to choose a reference element to be repeated periodically. In the case of irregular masonry, the frequency of any adopted reference element is hardly achieved, which makes this model unsuitable for this type of material.

In terms of applicability, it is a type of modelling clearly suitable when factors such as time, simplicity of modelling and computational capacity are crucial. It is a modelling strategy oriented for the everyday use in the analysis of real structures and when there is the need of maintaining a balance between accuracy and speed / efficiency, (Lourenço 2002).

A drawback of the macro-modelling approach lays in its description of damage as a smeared property spreading over a large volume of the structure. In real unreinforced masonry structures, damage appears normally localized in isolate large cracks or similar concentrated lesions. A smeared modelling of damage provides a rather unrealistic description of damage and may result in predictions either inaccurate or difficult to associate with real observations.

6.4.2 The modelling of Earthen structures using the Finite Element Method

6.4.2.1 Limitations of the Linear Finite Element Modelling

Linear finite element modelling is the only modelling technique known by authors to have been used to model the performance, both static and dynamic, of earthen materials. However, it is a field where one cannot really speak of a "state-of-the-art". For instance, the numerical modelling of cob is not even known to be present in the literature, whereas attempts at modelling rammed earth and adobe often do not take into account the homogeneous nature of the materials. In the case of rammed earth, this heterogeneous nature is due to the presence of planes corresponding to compression joints and horizontal joints at chases, (joints between formwork layers, sometimes filled with lime or rubble and characterised by holes), whereas in the case of adobe, this relates to the presence of vertical and horizontal mortar joints. One exception is Jaquin et al. (2006), who modelled rammed earth with interface elements.

Jaquin attempted to create a constitutive model for rammed earth based on an elastic-perfectly plastic continuum approach using the Mohr-Coulomb Failure Criterion. His model, whereby rammed earth is represented two layers, one layer representing the soil mass, and the other the interface, is non-homogeneous. Shear box laboratory tests and tri-axial cell tests were used to define soil properties. Although the model does not take the presence of a third joint in historic structures into consideration, i.e. the joints between formwork lifts, which are similar to mortar joints, the study is interesting because it argues that modelling of rammed earth can be geotechnical in nature, as opposed to purely structural.

Adobe masonry, which consists of blocks connected by mortar joints, is also a heterogeneous material.

However, in (Jäger, 2010), the properties of the masonry units were used to model the masonry itself, without taking the effect and properties of the mortar into account.

When used properly, (i.e. when the input values are correct), linear elastic models can give useful preliminary information about the seismic behavior and vulnerability of a building, in particular

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about its vibration modes, weak zones or about its elements with undesirable behavior, (Meli 2004) and are therefore a useful analytical tool. Another positive aspect is that it requires little input data, and is not so demanding in terms of computer resources or engineering time used if compared, for instance, to non-linear methods. The fact that little input data is required is particularly useful in the case of earthen materials, since it is normally only the compressive strength, density and young´s modulus that are normally known in earthen materials.

However, since joints in adobe masonry possess very low or even no-tensile strength, the assumption of elastic behavior is quite debatable, and therefore the grounds of Finite Element modelling on earthen construction so far are feasible. Modelling of earthen structures could be enhanced by applying "state-of-the-art" research in brick and stone masonry modelling to earthen construction in order to create reliable constitutive models.

In addition to elastic modelling, brick and stone masonry structures are most commonly idealised by means of limit modelling, (Orduña 2002), or more sophisticated methods, such as non-linear modelling. More recently, distinct element modelling has also been used to model historic masonry structures, (Azevedo 2001).

Plastic limit behavior in the framework of masonry structures is based on three fundamental assumptions: that the compressive strength of the constitutive material is infinite; that the tensile strength of the constitutive material zero, and that sliding between blocks along an interface is impossible.

While these assumptions can be justified in the case of brick and stone masonry, this is not the case for earthen materials, where compressive strength is very low (0.5 to4.0MPa), and tensile strength can of both units and mortar, due to the presence of fibres, can be relatively high (see flexural strength values from (Ziegert 2003)).

Non-linear Finite Element Method (FEM) and Discrete Element Method are therefore considered to be the two most viable options for the improvement of the behavioral understanding of earthen structures in earthquakes.

The feasibility of carrying out either technique, however, is limited, in that the material parameters known, (compressive strength, young´s Modulus, density), which suffice as input to linear elastic analysis, would not suffice for non-linear nor discrete element.

6.4.2.2 Non-Linear Approach

By adopting non-linear analysis, a clear insight into the structural response of earthen structures could be obtained at a higher computational and experimental cost. This extra cost is due to the fact that the construction of reliable advanced linear non-numerical models depends on advanced material characterization.

Numerous non-linear models for continuum and discontinuum approaches (Dhanasekar 1985), (Stankowski 1993), (Lofti 1994), (Pegon 1996), (Lourenço 1996), (Lourenço 1997), (Lourenço 1998a), (Oliveira 2003) have been developed for masonry. The latter, compared to the others, considers the modelling of cyclic modelling, as opposed to simply monotonic, incorporating stiffness degradation and energy dissipation.

Modelling of non-homogenous earthen materials, as per brick and stone masonry, (adobe and, to an extent, rammed earth) could therefore be carried out by means of (Rots 2008):

- Joints represented by continuum elements;
- Joints represented by discontinuum elements;
- Considering the masonry to be an anisotropic composite where joints and cracks are smeared out.

The issue of finding a compromise between micro- and macro-modelling as with other types of masonry is tricky.

While adobe can only be modelled as a homogeneous material after proper and thorough

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homogenization studies have taken place, (which still needs to occur), cob can be considered to be a homogeneous material.

6.5   DISCRETE ELEMENT METHOD

Apart from the finite element method (FEM) there are other ways of addressing the modelling problem, which have shown promising results, such as the discrete elements method (DEM). The Discrete Element Modelling is a discontinuum analysis technique which allows simulating progressive failure, crack propagation and large displacements/rotations between blocks. Compared to FEM, elements are able to move separately because they are not based on continuum mechanics In addition, by using DEM, the problem of interlocking is overcome since corners are automatically rounded. These are all features which make DEM very suitable for masonry structures, (Azevedo 2001).

The Discrete element method (DEM) is characterized by the modelling of the material as an assemblage of distinct blocks interacting along the boundaries. This method was originally applied by Cundall, (Cundall 1971), in the field of rock mechanics and as such, very useful for modelling materials such as masonry, which in many aspects is similar to rocks. According to this pioneer proposer, the name "discrete element" applies to a computer approach only if (1) it allows finite displacements and rotations of discrete bodies, including the complete detachment and (2) it can recognize new contacts between blocks automatically as the calculation progresses.

The formulation, initially oriented to the study of jointed rock, was later extended to other engineering applications also requiring a detailed study of contact between blocks or particles, such as soils and other granular materials, (Ghaboussi J. Barbosa R. 1990). Finally, it has also been applied to the modelling of masonry structures (Pagnoni 1994; Lemos 1998; Sincraian 2001). The common idea in the different applications of the discrete element method to masonry is the idealization of the material as a discontinuum where joints are modeled as contact surfaces between different blocks. This approach affords the modelling of various sources of non-linear behavior, including large displacements, and suits the study of failures in both the quasi static and dynamic ranges. The first applications of this type of element, to stone masonry structures, had as base the typical formulations of discrete models presented below:

- Rigid or deformable blocks. On the deformable blocks a finite element mesh can be considered;
- Contact conditions called "soft contact", in which the tensions are obtained from the relative displacement between blocks, taking into account the adopted normal and tangential properties. This type of contact model allows a slight overlap of blocks in compression;
- Explicit resolution of static and dynamic problems.

Other contact models are the "rigid contact", embodied in the work of authors such as (Jean 1995) and Acay and Jean (Acary et al. 1998) or also the contact models that resort to the use of springs on the surface of the blocks, (Casolo 2004a).

Distinct element methods, discrete-finite elements and discontinuous deformation analysis are different formulations of the discrete element method with important applications to masonry structures.

Distinct element methods are direct derivations of the first work by Cundall and Hart. They involve soft contact formulations where a normal interpenetration is needed to recognize contact between two different bodies. The main features of these methods are: (1) No restriction of block shapes and no limitation to the magnitudes of translational and rotational displacements. The approximation that all the deformations occur at the surfaces of blocks is made. It is also assumed that forces arise only at contacts between a corner and an edge. (2) Forces arise due to deformation. A change in displacement results in a change in force which is added to the existing force stored for the contact. (3) Accelerations are computed from the forces and moment for each

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block. The accelerations are further integrated into velocity and displacements. (4) Contact updating is performed when the sum of the displacements of all of the elements has exceeded a certain value. To increase the efficiency, only blocks within a certain distance range are checked for new contacts.

As in the case of finite element, modelling masonry with discrete elements can be performed at the micro or macro level. Based on the presented modelling philosophy, the application of discrete elements at the micro level has clear advantages, especially in what regards the ease of modelling and computational requirements. In this type of approach, at a micro level, the main difference between the finite element method and the discrete elements method regards the way it is modeled the contact between the different elements; on the modelling using the finite element method it is considered an interface surface (ex. joint models) while on the modelling with discrete elements, the contact is done through contact points that allow analysis with large displacements. However, despite the existing differences between these two approaches, their development tends to bring them together, arriving to a point in which these approaches complement each other. There are also hybrid modelling solutions, called finite/discrete element method, that appear on the work of authors such as Munjiza in (Munjiza 2004), Petrinic in (Petrinic 1996) or Barbosa in (Barbosa 1996), as described by Viera de Lemos in (Lemos 2007).

Discrete-finite element methods recollect different attempts of combining FEM with multi-body dynamics. Munjiza et al. (1995) developed a method for the simulation of fracturing problems considering deformable blocks that may split and separate during the analysis. Mamaghani et al. (1999) used a fixed contact system with a small deformation framework and finite deformations concentrated in contact elements. Contacts, discontinuities and interfaces were considered as bands with a finite thickness. The contact element was a two-nodded element having normal and shear stiffness's. The method was applied to the stability analysis of different masonry structures.

The natural field of application of DEMs is composed by structures formed by regularly shaped masonry or stone blocks. Rocking motion of stone blocks (Peña et al. 2007), static and dynamic analysis of load bearing walls (Pagnoni 1994; Baggio and Trovalusci 1995; Schlegel and Rautenstrauch 2004), stone bridges (Lemos 1995; Bicanic et al 2001), columns and architrave (Papastamatiou and Psycharis 1993; Psycharis et al 2003, **Errore. L'origine riferimento non è stata trovata.**), arch and pillar (Pagnoni 1994; Pagnoni and Vanzi 1995; Lemos 1998) are typical examples of DEM analysis. The analyses of complex structures is still a controversial topic in DEM. Computational viability of analysis may limit severely the number of block elements that can be included in a model. Models prepared to simulate the response of real structures may result in too coarse or unrealistic discretizations or 2D, and specially, 3D real masonry structures.
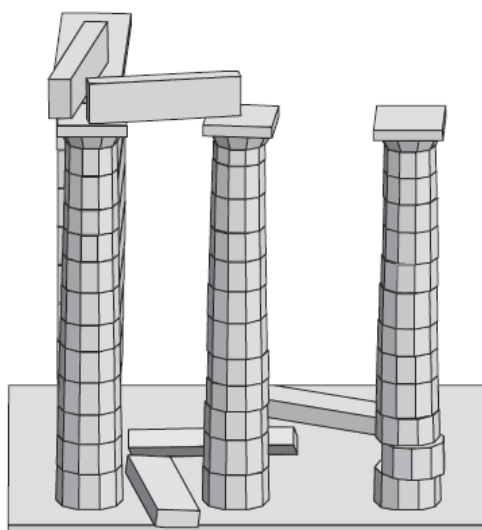

Figure 6.14 Final position of the column-architrave model of the Parthenon Pronaos, without reinforcement, for a

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simulated earthquake, (Psycharis et al. 2003).

In what regards the macro modelling, the discretization of a material through a continuum homogenized using a discrete element method does not make sense, since, besides being necessary to apply finite element modelling the capabilities of the discrete elements programs are limited in this area when compared to the finite element programs. The use of discrete elements in the field of macro modelling has advantages when using approaches such as that proposed by Siro Casolo in (Casolo, 2004b). In this work the author proposes a modelling methodology for regular masonry, in which the behavior of a portion of masonry is homogenized and numerically simulated by a reference unit, composed of rigid quadrilateral elements connected by two normal springs and a shear spring, whose characteristics are defined by considering the effects of texture that arise due to mechanical degradation of the infill.

In this approach, the masonry (heterogeneous composite material) is treated as a continuum, not involving however, the homogenization of the entire structure in a continuum and thus avoiding the complexities related to the assignment of elastic and plastic characteristics to that same continuum. This is achieved by fully renouncing the continuum concept and exploiting the intrinsic characteristics of the rigid elements kinematic. In addition, this type of approach requires a reduced computational effort, which is promising when more complex numerical modelling is in mind, such as the nonlinear dynamic analysis.

More recently and in sequence of the work described above, Siro Casolo and Fernando Peña, (Casolo, 2007), propose a strategy for numerical modelling with rigid elements, for the in-plane dynamic study of irregular masonry walls whose seismic response is strongly correlated with mechanical deterioration and hysteretic energy dissipation.

To this end, on this work a spring model was adopted, which consists of rigid elements connected by springs, similar to that described above, in addition, the used material model was based on a phenomenological description of the masonry cyclic response, having been assigned hysteretic laws specific and independent to the axial and shear deformation between elements. This separation, according to the author, led to a big reduction in computational effort; despite having been used a Coulomb-type law in order to relate the shear springs effort with the axial vertical load.

As part of the ISTECH project, Azevedo and Sincraian (Azevedo 20012001) successfully analysed masonry by means of a 2D discrete model analysis technique. The necessary parameters to define the contacts mechanical behavior were normal stiffness, shear stiffness, friction angle, cohesion and tensile strength. The study concluded that the discrete element method can lead to a correct representation of the failure mechanisms of masonry structures during earthquakes.

6.6 MODELLING PROBLEMATIC

To analyze the behavior of historical structures there are nowadays, several methods and computational tools that are based on different theories and strategies, resulting in different levels of complexity (from simple graphical methods and hand calculations to complex mathematical formulations and extensive non-linear systems equations), different calculation times and, of course, different costs.

When opting for one method of analysis one must have clearly defined the desired type of analysis, its objectives and also the knowledge of the advantages and limitations of the available tools, bearing in mind that more complex analysis are not necessarily synonymous of better results, (Lourenço 2002). Above all, an analysis must be informed and planned in order to maximize its simplicity.

A practical analysis of historical structures implies great simplifications in the creation of the model geometry; the technician responsible for the analysis has to assess what is or is not important for a given analysis. The geometric idealization can and should be kept as simple as possible, as long as it is considered appropriate to resolve the problem.

Despite nowadays the technological and scientific development permits the execution of increasingly complex analysis in increasingly shorter times, often increasing the detail and size of a

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model can create difficulties on the analyze of the structure behavior and consequently of the results, originating a loss of objectivity and resulting on a huge amount of information which may not be specific on the eventual detail which is intended to be analyzed, (Silva et al. 2008a).

Whenever possible, it is preferable to individually analyze the elements of a structure, simplifying its geometry and reducing the computation effort. In this case it may be advisable to use 2D models instead of the 3D ones. In cases where there is interest, or it is considered important to use the complete model of the structure it is advised the use of mixed complexity, detailing (from the geometrical and (or) material behavior point of view) the areas of major interest (with higher influence on the overall behavior of the structure, or critical areas that present particular problems to be analyzed).

In what regards modelling with finite elements, (currently the most "popular"), continuous rigid elements, as the case of masonry walls, it is frequent to use shell elements, since these present advantages in terms of ease of creation of the geometrical model and of computational effort. However, certain precautions need to be taken into account and several difficulties arise when using this type of element, such as:

- The use of this type of element does not allow considering, in a direct manner, the eccentricity, since the shells are aligned to the axis. In cases where there are large variations in thickness on a structure, it is important to take into account this phenomenon, in particular, when the floors are supported only in one of the leaves of a multiple leaf wall panel.

- The use of such elements makes it impossible to consider directly the phenomenon of leaves separation, i.e. it can only be used when this type of phenomenon is not important to analyze the problem, or when the structural element in question is monolithic.

- In finite elements, the stress distribution along the thickness of a wall is linear, which may deviate from reality, at a local level.

- In such elements, the stiffness in the areas of intersection of two shells (ex: facade angles) is not always well represented.

The use of volumetric elements allows reproducing, in a more realistic way, the intersection zones of structural elements. By using this type of modelling it is possible to evaluate the stresses in the thickness of a wall, however, more than one element to discretize the mesh along the thickness as to be used, otherwise the errors will be high, and the greater the thickness of the actual element the greater the error. In contrast, the use of volumetric elements makes the creation of the geometric model more complex and time consuming.

The modelling of the links between elements of a structure is almost always one of the most important problems of numerical modelling of historical structures. This difficulty arises not only from the difficulty in defining them appropriately, but also from the choice of numerical models capable of realistically characterize these links. For a question of simplicity, in most cases the connections are considered as continuous and fixed, but this type of assumption is only valid in certain cases and it is the responsibility of the technician in charge of the analysis to evaluate this assumption in each case.

In the particular case of the support conditions of a structure to the soil, most of the times the structure is considered as fixed at the base. However, this assumption is only valid in the cases where the soil presents good quality and the wall is properly fixed in the soil. Otherwise, the soil as to be modeled as well or in alternative the soil can be modeled for example with springs at the base with stiffness characteristics equivalent to the ones presented by the considered type of soil.

In the modelling of wooden roofs reinforced with metal rods only the tensile resistance must be considered on the ties, using for that appropriated models and in order to make the analysis more realistic.

Given the large number of parameters of which depends the analysis of historical structures and the degree of uncertainty and lack of knowledge that surrounds them, it is not appropriate and / or

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correct to propose a single method of analysis. In addition, on the process of choosing a method of analysis one must take into consideration the limitations and advantages of each method and the objectives of the analysis. As such, it is correct to propose an analysis of a same problem using different methods, and the comparison between the results of each method can increase the degree of confidence in the obtained results. Several authors recognized in the area are an example of the combined application of different modelling strategies, on the analysis of this type of structures, such as the limit analysis and numerical modelling, (Lourenço 2001), (Orduña et al. 2001), (Brencich et al. 2001), (Casarin 2006), Figure 6.15. The numerical models allow individualizing macro-elements and their respective lines of collapse and of mechanisms formation of a structure when subjected to seismic action, whose vulnerability and safety can be analyzed through simplified limit analysis.
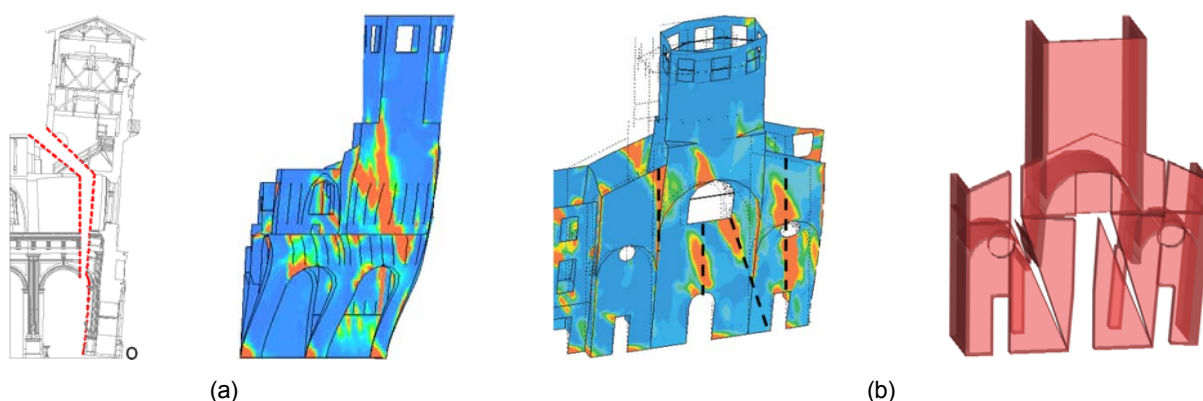

(a)　　　　　　　　　　　　　　　　　　　　　　　(b)

Figure 6.15 Comparative analyzes, using different analysis methods, of the Cathedral of Santa Maria Assunta, Reggio Emilia (Italy), (Casarin 2006). (a) Out-of-plane mechanism of the frontal façade. (b) In-plane mechanism of the frontal façade.

It is of great importance to validate the more complex models with simpler analytical calculations, such as the simple calculation of the weight of a structural element and the confrontation with the reactions obtained at the base with the numerical model, or even the analytical calculation of the compressive stress due to the structure self-weight in a given point and confront them with the tensions obtained by the numerical model.

The FE model of an historic structure always involves simplifying assumptions and several uncertainties in the material, the geometric properties and boundary conditions, even if accurately surveyed. Within this context, one possible key role of Operational Modal Analysis is to provide an effective and accurate validation of the model prior to its use in numerical analysis, (Gentile 2007, 2010).

6.7   MODELLING OF STRENGTHENED STRUCTURES

Structural modelling may contribute to the validation of possible strengthening techniques by simulating their effect on an accurate numerical model of the structure. The analysis of the strengthened structure requires, in particular, the modelling of the strengthening techniques implemented. For that purpose, it may be useful to model the possible strengthening techniques by the combination of the basic actions they produce in the structure. These basic actions have been referred in section 1, while the actions produced by any strengthening technique are (tentatively) mentioned for each case in section 2. Once the basic actions are identified, they can be introduced in the numerical model as a combination of some essential numerical devices. The main numerical devices which can be used to model possible strengthening actions are:

1. _Creation of an internal constraint:_ connection two or more nodes of the structure with a stiff or rigid element, thus limiting the reciprocal displacement. The strength and stiffness of the

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connection has to be carefully investigated. It can be used to model *confinement*, *tying* and *reinforcement,* (seen as high resistance material well connected with the original one).

2. *Improvement of the mechanical properties of the material:* modification of the material properties of the masonry, (in a macro-model), or in the joints or the blocks, (in a micro-model). It can model all the techniques involving a removal and replacement of parts of a structure as well as general improvement of the material masonry. This device can be utilized to model *material substitution* and *improvement*.

3. *Inclusion of an additional substructure:* modelling of new structures interacting with the original ones. It consists simply in superposing two finite elements model. It can be used to model *structural substitution* and *propping*.

4. *Creation of an external constraint:* connection of one or more nodes of the structure to an external element or to a fixed node. It can model *anchoring*.

5. *Application of loads:* application of external nodal or distributed forces to the structure. This device can be used to model *pre-stressing*.

6. *Widening of the section:* increasing of the physical dimensions of the resisting section. This resource is appropriate for the numerical modelling of *enlargement*.

7. *Modification of the seismic action:* decrease or modification of the seismic forces or accelerations applied to the structure. It can be used to model *isolation* and *soil stabilization*.

Note that each device refers to one or more basic actions. Thus, any strengthening technique can be described in terms of the series of actions it cause. In turn, such actions can be modelled as a combination of the aforementioned numerical devices.

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