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Development of innovative methods for the seismic risk classification of masonry aggregates

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Statement of integrity

I hereby declare having conducted my thesis with integrity. I confirm that I have not used plagiarism or any form of falsification of results in the process of the thesis elaboration.

I further declare that I have fully acknowledge the Coode of Ethical Conduct of the International PhD Course Civil and Environmental Engineering of the University of Florence and of the University of Minho.

University of Pisa, December 31th 2020

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Signature:

A handwritten signature in blue ink that reads "Federico Romis". The signature is written in a cursive style with a large, stylized 'F' and 'R'.

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Abstract

Historical centres are the result of a process occurred during the centuries: masonry buildings constituting the urban environment are usually organized within Structural Aggregates (SAs) made up of Structural Units (SUs), connected in a variety of different ways, characterized by heterogeneous structural features, materials and architectures. The morphological variety of the resulting urban settings enriches the cultural heritage of a place, meanwhile, increases local and global vulnerabilities towards static and seismic actions. Consequently, the study of transformation processes of a historic centre is the first step for the prevention and mitigation of seismic risk. Another important assumption, as result of the first one, is that the protection of historical centres requires many resources in terms of assessing the seismic vulnerability and implementing retrofit strategies considering each individual building and their mutual interaction. In this sense, the mitigation of the seismic risk of historical built heritage has to be performed through successive in-depth process: firstly, the identification of the most vulnerable aggregates by means of urban scale screening; secondly detailed analyses on these aggregates to identify specific critical issues.

The aim of this research activity is to provide new tools for the analyses of seismic risk on urban scale, in order to forecast damage scenarios and economic impact of seismic events on masonry aggregates: four clear objectives are accomplished through the development of four research tasks structured along the eight chapters that compose this document. The first part of the thesis provides a general overview of all those necessary procedures for a correct identification of the aggregate and its corresponding seismic vulnerability assessment. The literature review and on-site inspection, carried out in the case study of Campi Alto di Norcia in Valnerina (Italy) stroke by the earthquakes of 24 August and 30 October 2016, highlight advantages and disadvantages of current approaches and tools used for the vulnerability assessment of masonry aggregates. The second part deals with the development of an improved Empirical /Expert base method for the seismic vulnerability assessment of SAs on urban scale. Indeed, different statistical methods are selected and applied to the case study with the aim of determining the corresponding vulnerability index (I_v) and comparing their accuracy in predicting damage scenarios. Since recent earthquakes have shown that local mechanisms often cause failures of masonry aggregates, new parameters are introduced in the Improved Method, to take into account the out of plane behaviour of structural units in aggregates.

The third part of the thesis addresses the calibration of the Improved method, by comparing the 'observed' damage caused by the 2016 earthquake in the case study with the 'analytical' ones predicted. To overcome the limits of the Empirical /Expert base methods, analysed in the previous phases, an additional Information Quality (IQ) index is, then, introduced in the calibrated procedure accounting for uncertainties related to the lack of information due to limited access or inspections. Finally, the work ends with the elaboration of codified procedure, identified with the acronym "MARS" (Masonry Aggregate Risk Scenarios), that takes into account damage, loss and economic impact of seismic events on masonry aggregates and provide further information for the seismic risk management of historic centres.

Keywords: Vulnerability index method, damage scenarios, masonry buildings, seismic assessment, structural aggregates, seismic risk, historic centre.

Resumo

Os núcleos urbanos antigos são o resultado dum processo que aconteceu ao longo dos séculos: os edifícios de alvenaria que constituem o ambiente urbano são normalmente organizados em Agregados Estruturais (SAs) constituídos por Unidades Estruturais (SUs), interligados de formas variadas, e com características estruturais, materiais e arquiteturas heterogéneas. A variedade morfológica das configurações urbanas torna estes espaços mais ricos do ponto de vista do seu valor cultural e patrimonial, aumentando, no entanto, as vulnerabilidades locais e globais face às ações sísmicas. Consequentemente, o estudo dos processos de transformação de um centro histórico é o primeiro passo com vista à prevenção e à mitigação do risco sísmico. Um outro aspeto importante, decorrente do primeiro, é que a proteção dos centros históricos requer uma quantidade significativa de recursos, quer para a avaliação da sua vulnerabilidade sísmica, quer na fase de implementação de estratégias de reforço, devendo cada edifício ser avaliado individualmente e enquanto parte do conjunto, i.e., considerando interação entre edifícios contíguos. Neste sentido, a mitigação do risco sísmico do património edificado deve ser efetuada através de um processo de análise bietápico: em primeiro lugar, com vista à identificação dos agregados mais vulneráveis através de observações à escala urbana; de seguida, com o intuito de avaliar detalhadamente os agregados de forma a identificar e avaliar as suas potenciais fragilidades.

No contexto acima descrito, o objetivo central desta investigação é fornecer novas ferramentas para a análise de risco sísmico à escala urbana, com o fim último de, a partir da análise da vulnerabilidade sísmica de agregados de edifícios, obter cenários de dano e estimar perdas económicas resultantes de eventos sísmicos: os quatro objetivos são alcançados através do desenvolvimento das quatro tarefas de investigação estruturadas ao longo dos oito capítulos. A primeira parte da tese oferece uma visão geral do conjunto de procedimentos necessários à correta identificação do agregado e sua correspondente avaliação de vulnerabilidade sísmica. A análise da literatura e a descrição da campanha de inspeção *in situ*, realizada no caso de estudo Campi Alto di Norcia em Valnerina (Itália), após os eventos sísmicos de 24 de agosto e 30 de outubro de 2016, destacam as vantagens e desvantagens das abordagens e ferramentas correntemente utilizadas para a vulnerabilidade avaliação de agregados de edifícios de alvenaria. A segunda parte trata do desenvolvimento de um método de base empírica para a avaliação da vulnerabilidade sísmica de SAs à escala urbana. Para tal, foram selecionados e aplicados ao caso de estudo um conjunto de métodos de avaliação da vulnerabilidade sísmica, com o objetivo de determinar o índice de vulnerabilidade correspondente (Iv) e comparar a sua precisão na previsão de cenários de danos. Com base nesse aplicação, e partindo das limitações reveladas por esses métodos, desenvolveu-se um Método Melhorado capaz de integrar e avaliar a vulnerabilidade global e local (comportamento fora do plano) de unidades estruturais em agregado. A terceira parte da tese aborda a calibração e validação do Método Melhorado, com base na comparação entre os danos ‘observados’ na sequência do terremoto de 2016 e os danos ‘analíticos’ obtidos através da aplicação do método. Para superar os limites dos métodos de base empírica, analisados nas fases anteriores, foi ainda proposto um índice de Qualidade da Informação (IQ) com o intuito de considerar as incertezas relacionadas com eventuais faltas de informação decorrentes de limitações no acesso aos edifícios durante o processo de inspeção. Por fim, o trabalho termina com a elaboração do procedimento codificado, identificado com a sigla “MARS” (Cenários de Risco nos Agregados de Alvenaria), que permite estimar danos e perdas (humanas e económicas), para cenários sísmicos determinados, e, com base nessa informação, apoiar estratégias de mitigação e gestão do risco sísmico à escala urbana.

Palavras-chave: Índice de vulnerabilidade sísmica, cenários de danos, edifícios de alvenaria, avaliação sísmica, agregados estruturais, risco sísmico, centro histórico

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CHAPTER 1. Introduction

The chapter provides a general overview of the motivations and objectives behind this research activity. It describes how the latter are achieved in the research methodology and how the work is organized within the thesis.

Chapter outline

- 1.1. Background and motivation*
- 1.2. Thesis objectives*
- 1.3. Research methodology*
- 1.4. Outline and organization of the thesis*
- 1.5 References*

1.1. Background and motivation

The earthquakes of the last decades that hit different European countries (Albania - M_w 6.6 - 26/11/2019) and above all the Italian territory (L'Aquila - M_w 6.3 - 06/04/2009, Emilia Romagna - M_w 5.9 - 29/05/2012, Accumoli – M_w 6.0 - 24/08/2016 and Norcia – M_w 6.5 - 30/10/2016), showed the necessity to deal with the recovery and seismic risk assessment of historic centres.

The historical heritage is the result of an interaction process between people and the surrounding area: the heterogeneous architecture of ancient city centres is the direct expression of cultural diversity, natural transformations and anthropic events occurred over the centuries. The masonry buildings constituting the urban environment are usually organized within *Structural Aggregates* (SAs), not following a well-scheduled development and therefore resulting in different construction typologies in relation to different locations and realization periods (Giuffrè 1993). SAs are typically made up of buildings – recognized as '*Structural Units*' (SU) – connected in a variety of different ways, characterized by heterogeneous structural features, materials and architectures: different configurations are possible for a SA, whose resulting structural performance is influenced by each single SU constituting it (Caprili et al. 2016).

The morphological variety of the resulting urban settings enriches the cultural heritage of a place, meanwhile, increases local and global vulnerabilities towards static and seismic actions. Recent earthquakes that struck Italy during the last decades (INGV 2019) highlighted failures of masonry aggregates caused also by the activation of local mechanisms, not allowing the development of global building behaviour (Dolce et al. 2004), (Formisano et al. 2010), (Borghini et al. 2011), together with the negative impact of past retrofit interventions that frequently increased vulnerabilities instead of reducing them (Borri et al. 2018).

There are, indeed, two recurrent types of collapses for this construction typology (Doglioni et al. 1994), (Cifani et al. 2008), (Monti 2009): the in-plane and the out-of-plane failure mechanisms. In this sense, damage has to be considered hybrid; influenced both by a local and a global behavior of masonry buildings. The seismic

vulnerability assessment of masonry building in aggregate must, therefore, takes into account these two effects, considering the presence or absence of structural features that cause the different failures.

The implementation of seismic prevention and retrofit strategies for historical city centres requires the knowledge, on a territorial scale, of the risk to which existing SAs are exposed (MIBACT 2010), of the adopted materials, structural features, morphological evolution of the aggregates and, above all, of the interaction between SUs and SA during the earthquake.

Different studies carried out following seismic events occurred in the recent years in Italy, show the number of casualties and the amount of economic resources spent on the management of emergency activities and the reconstruction of towns and historic centres (Bellicini et al. 2012), (Centro Studi Consiglio Nazionale Ingegneri 2014). In this sense, new approaches have been developed for the mitigation of seismic risk, introducing the seismic classification of existing buildings, to take into account economic and human losses through risk classes (DM n. 58 2017).

Seismic risk is the combination of structural vulnerability, exposure and seismic hazard; the analysis of each of the three above-mentioned parameters become a relevant issue for the global understanding of the state of the art of an historical construction or SA. Several methods exist in the current scientific literature for the vulnerability assessment of masonry aggregates in historical city centres, based on a multi-level approach characterized by increasing knowledge and in-depth in-situ evaluations (Cattari et al. 2006).

Among them, the simplest approaches are known as statistical (or observational) methods; they can be used for a quick and easy analysis of seismic risk at regional/urban scale by defining, for each masonry construction, a *Vulnerability Index* (I_v) accounting for different parameters whose weight is assumed basing on expert evaluations of the post-seismic damage conditions (Ortega et al. 2018). The statistical approach allows to summarize the observed damages and the results of in-situ investigations through Damage Probability Matrices (DPMs), analysing vulnerabilities and estimating damage for different structural typologies (D'Ayala et al. 1997), (Cattari et al. 2004). Statistical methods are powerful tools for providing a first screening of constructions' fragilities on the territory, determining a sort of priority list for the execution of deeper investigations and for the following planning of retrofit interventions. Their limitation lies in the constructions' scenario to which they can be applied: their reliability progressively decreases when applied to structures considerably different respect to the ones used for the method's calibration (Ferreira et al. 2017). Besides, the lack of information coming from a survey may overlook critical epistemic uncertainties, influencing the results (Novelli et al. 2015).

Basing on what herein stated, the protection of historical centres towards seismic risk requires many resources in terms assessing seismic vulnerability and implementing retrofit strategies considering each individual building and their mutual interaction.

1.2. Thesis objectives

The present PhD research intends to develop a procedure, identified with the acronym "MARS" (*Masonry Aggregate Risk Scenarios*), for the seismic risk class estimation of masonry aggregates and, in general, of historical city centres characterized by high cultural and historical value, which are not provided by adequate analysis' tools for achieving reliable large-scale vulnerability assessments.

The proposed procedure is based on an improved statistical/observational methodology, where the in-plane and the out-of-plane failure mechanisms are considered. The aim of this research activity is, therefore, focused on providing a first screening of constructions' fragilities on the territory, determining a priority list for the execution of deeper investigations and for the following planning of retrofit interventions (Kassem et al. 2020).

In this sense, the research is outlined around four specific and fundamental objectives:

- **OBJECTIVE 1:** General overview on the determination of the seismic vulnerability of Structural Units inside the Structural Aggregate, through literature review, historical analysis and morphological evolution, analysis of past damages, typological and structural features, classification of main vulnerabilities and evaluation of interconnections;
- **OBJECTIVE 2:** Application of different statistical methods to a case study, with the aim of determining the corresponding vulnerability index (I_v) and comparing their accuracy in predicting the damage scenarios;
- **OBJECTIVE 3:** Development of a seismic vulnerability assessment method for masonry aggregate in historical centres, taking into account the in-plane, as well as the out-of-plane behaviour;
- **OBJECTIVE 4:** Elaboration of codified document that takes into account damage, loss and economic impact of seismic events on historical centers (urban scale).

1.3. Research methodology

The objectives of the research are achieved by adopting different approaches developed in the scientific literature for the data collection and analysis, for the damage observations, for the historical and structural understanding and for the statistical analysis.

The main core of the work is the development of an *Improved Method*, according to the vulnerability index methodology, for the seismic vulnerability assessment of SUs in masonry aggregate. Respect to other methods presented in the current literature, the proposed one accounts for the out-of-plane and in-plane behaviour. Besides, the lack of information about specific parameters and the influence coming from the experience of the technician performing the survey is overcome, by introducing an Information Quality (IQ) index. In this sense, for each analysed building a vulnerability index range is evaluated, from which different possible damage scenarios are defined.

A multi-step approach is, therefore, adopted during the research activity, owning the advantage of being calibrated basing on the damages really observed in the case study of Campi Alto di Norcia, Umbria (Italy), after the earthquake crises of Central Italy (2016): This makes the proposed procedure reliable for a variety of construction typologies diffused on the territory in Central Italy (Benetti et al. 1998), reducing the error connected to the application of traditional literature approaches.

Considering several vulnerability indexes, fragility curves are developed and evaluated for different seismic intensities, based on probability ranges of damage classes, according to the EMS Damage Scale (Grünthal 1998). With the aim to predict the expected damage on large scale, a correlation between seismic intensity and peak ground acceleration is introduced (Faenza and Michelini 2010).

Basing on the range of the vulnerability index defined for the SU, analysed with the *Improved Method*, and the seismic hazard of the place where the SU is located, the seismic risk class of the SU is determined, by calibrating the economic losses with the damage detected in the case study.

The study is divided in four different phases to fulfil the thesis objectives, briefly summarized in the following, see Figure 1.1.

PHASE 1: STATE OF THE ART AND CRITICAL ANALYSIS

The first phase provides the analysis and regulation of all those procedures necessary for a correct identification of the aggregate and its seismic vulnerability assessment, that, starting from a diagnosis and survey process, allows to examine the parameters that most influence the dynamic response of a SU within the SA, including structural and non-structural components (e.g. balconies and chimneys). These procedures will be the summary of the methodologies developed over the years, to evaluate the seismic vulnerability and transformations that have affected the masonry aggregates, assumed as the principal cause of the criticalities and vulnerabilities of the historical city centres. The advantages and disadvantages of current approaches and tools used for the vulnerability assessment are examined and compared, in relation to the analysis of masonry buildings in aggregate. The *literature review* and on-site inspections, carried out in the case study, provides information for the elaboration of the research activity.

PHASE 2: DEVELOPMENT OF AN IMPROVED *lv* METHOD

The second phase is the main objective of the study and is directly related to the second and third objectives of the research: different statistical methods are selected and applied to Campi Alto di Norcia (Italy) with the aim of determining the corresponding *lv* and comparing their accuracy in predicting damage scenarios. The purpose of this activity is to highlight possible limitations of selected methods, finally choosing the most performing one for the considered case study. A database of information with data coming from in-situ surveys is elaborated.

Basing on the results and experience achieved, new vulnerability parameters are introduced and some other parameters of the most reliable method are redefined, in order to take into account in an *Improved Method*, the out of plane behavior of structural units in aggregates. The construction typology of the case study of Campi Alto di Norcia is used as benchmark for the development of different façades configurations, in order to perform different parametric numerical analysis. The aim is the assessment and quantification of the influence of the different parameters in the out of plane behaviour of SUs in aggregate. The numerical evaluation is based on the rigid body limit analysis considering the rocking mechanism of different façade configurations, starting from the façade benchmark. The parametric analysis is performed in terms of capacity curve obtained by an incremental kinematic approach, following the upper bound limit analysis.

Moreover, statistical analyses are executed to arrange the façade configurations that lead to failure and to define the vulnerability parameter classes through the variation of the collapsed load multiplier. Guidelines are at least developed for the correct scores assignment of the new vulnerability parameters, according to the vulnerability index methodology.

PHASE 3: CALIBRATION AND OPTIMIZATION OF THE IMPROVED I_v METHOD

In the third phase, the Improved Method is tested in the case study for the seismic vulnerability assessment of the masonry buildings of the historical centre. The weights of the Improved Method are calibrated, by comparing the '*observed*' damages caused by the 2016 earthquake in the case study with the '*analytical*' ones predicted through the (Bernardini et al. 2007) formulation. Accurate literature review and analysis are performed, for the individuation of the *calibration systems*, employed in the engineering field, suitable for the research problem.

An additional IQ index is, then, introduced in the calibrated procedure accounting for uncertainties related to the lack of information due to limited access or inspections. Confidence Level (CL) classes are defined for the different vulnerability parameters used in the Improved method. In relation to assessed CL, a coefficient accounting for related uncertainties (UL) is determined and applied to the weight of each vulnerability parameter, determining the relevance of the parameter for the seismic behaviour of the building. The weighted average of the UL defines the IQ and is used to increase or reduce the I_v .

In this sense, the Vulnerability Index (I_v) of SUs in aggregate, evaluated through the Improved Method, is defined in a range of possible values limited by two boundary conditions, where the vulnerability index expresses an accurate result. The use of ranges instead of single values for the seismic vulnerability assessment of masonry buildings, adopting these observational methods, is considered a step forward in the contribution for scientific knowledge.

PHASE 4: SEISMIC RISK CLASSIFICATION

The final part of the research activity is dedicated to identification of seismic risk classification for masonry buildings in aggregate. The results represent the purpose of the last objective of the research, as well as the translation into codified tool of the work carried out, for the elaboration of the *MARS procedure*.

A probabilistic approach is used for the development of statistical analyses, in order to evaluate probability damage ranges for the construction typology of the case study.

In this sense, vulnerability classes are defined in relation to different vulnerability index ranges. With the latter fragility curves are developed and physical damage scenarios of the analysed building stock are estimated for several seismic intensities. Probability ranges of different damage levels are defined, according to the EMS – 98 Damage Scale. Since the seismic intensity is based on the observed damage after a seismic event, a correlation between the seismic intensity and the pga is introduced, in order to link the seismic hazard with the location of SUs. The probable damage can be, therefore, predicted prior the event.

Economic losses are then evaluated based on the fragility curves developed for specific damage levels, and calibrated with losses recorded in Campi Alto di Norcia, after the 2016 earthquake.

Finally, seismic risk classes are associated to the damage probability in relation to the seismic hazard and the Vulnerability Index Ranges. A damage Probability matrix is introduced to identify the seismic risk class for a specific building in a historical Centre, through an observational method.

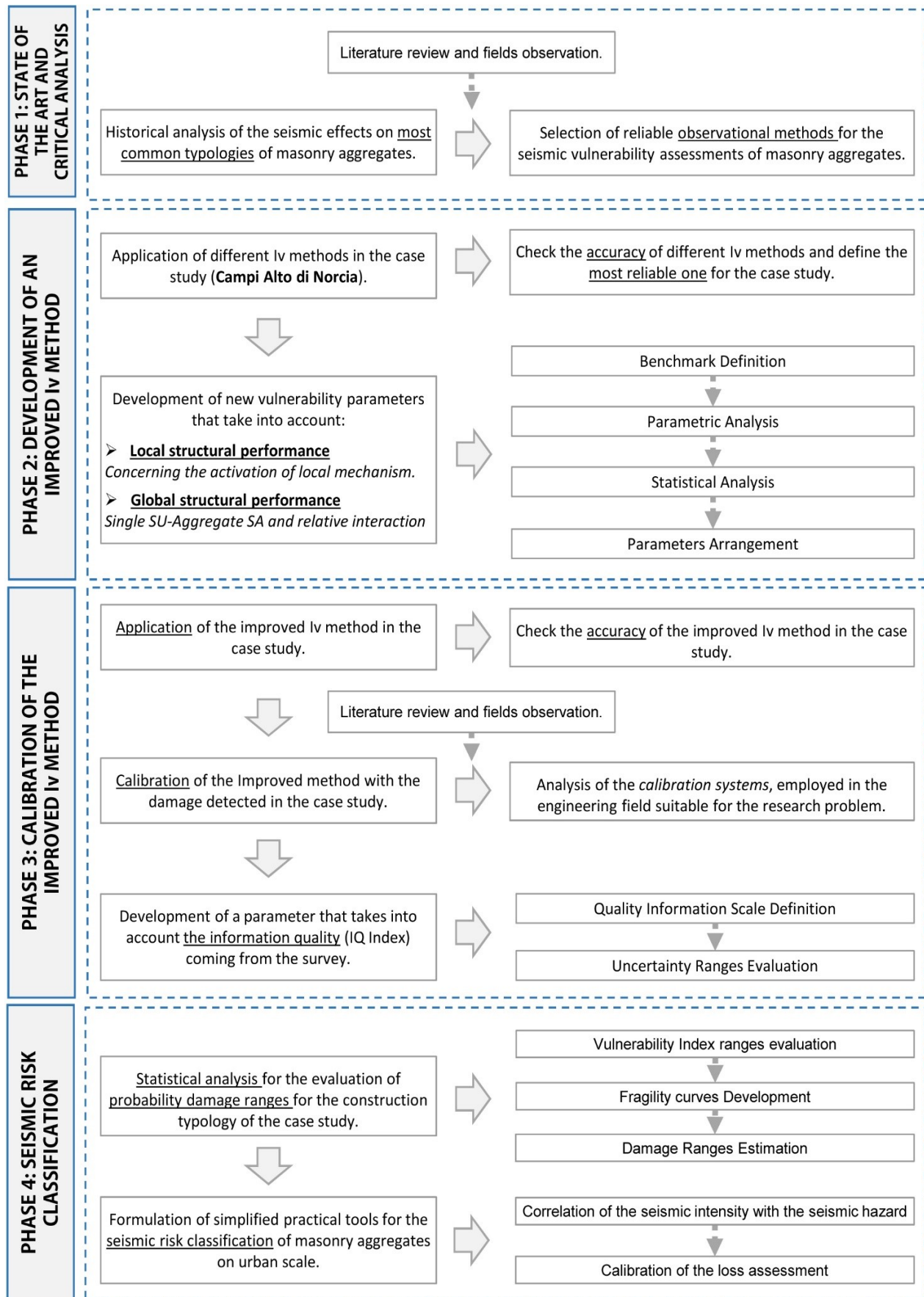


Figure 1.1: Development of the research activity (methodology)

1.4. Outline and organization of the thesis

This work is organized in eight different chapters and annexes (Figure 1.2). The present chapter is an introduction of the work performed and provides a general overview on the objectives and motivation behind the research activity. To achieve the objectives of the research, a methodology is developed, considering different phases, described in detail in the chapters of the thesis.

In the second chapter a review of the principal procedures for the seismic risk assessment of existing buildings is exposed. The purpose of this chapter is to compare and to analyse the different methods to show their advantages and disadvantages. Finally, four methods regarding the vulnerability index methodology are fully described, highlighting their characteristics and the formulations adopted, which will be used throughout the rest of the work.

The third chapter contains the analyses of the historical centre of the case study of Campi Alto di Norcia. The geometrical and structural morphology of the current SAs is assessed through the knowledge of the historical background, the analyses of the materials used for the construction of the buildings and the study of urban organization. In this sense, the structural performance of the construction typology of the case study is defined, considered representative of several historical centres diffused on the Apennines of Central Italy.

The characteristics of the 2016 Central Italy earthquake and the damage surveyed in Campi Alto di Norcia after the seismic event are described in chapter four. The information contained in the latter represent the database on which the calibration procedure of the improved method is based.

In the fifth chapter, the selected vulnerability methods are applied in the case study of Campi Alti di Norcia. In this sense, the principal features of these methods are shown, as well as their accuracy in predicting the damage scenarios.

Based on the most reliable vulnerability index method for the construction typology of the case study, in chapter six, new vulnerability parameters are introduced and some other parameters are redefined, in order to develop an Improved Method. In this sense, the weights of the Improved Method are calibrated with the damage detected in the case study through an innovative calibration procedure and the IQ Index is defined for the evaluation of different damage scenarios. Moreover, the weights of the parameters are compared with the results obtained by collecting the opinion of experts in the field through a survey. The questionnaire established for the survey is shown in Annex D. The variations among the weights defined with the mathematical/statistical analysis and the results from the expert opinion are critically discussed.

In chapter seven the MARS (Masonry Aggregates Risk Scenarios) procedure is presented, through the evaluation of the seismic risk classification on urban scale of masonry buildings in aggregate. This procedure is applied, calibrated and validated in the case study of Campi Alto di Norcia to define the seismic risk class, considering the seismic hazard of the surrounding area.

The last chapter summarizes the work performed, presenting the results obtained and the conclusions achieved. Furthermore, it contains suggestions and recommendation for future improvement of the procedure and the seismic vulnerability assessment method. In this sense, it is possible to increase the sample space of the data used for the statistical analysis, taking into account further seismic intensities and considering other formulations for the definition of damage scenarios.

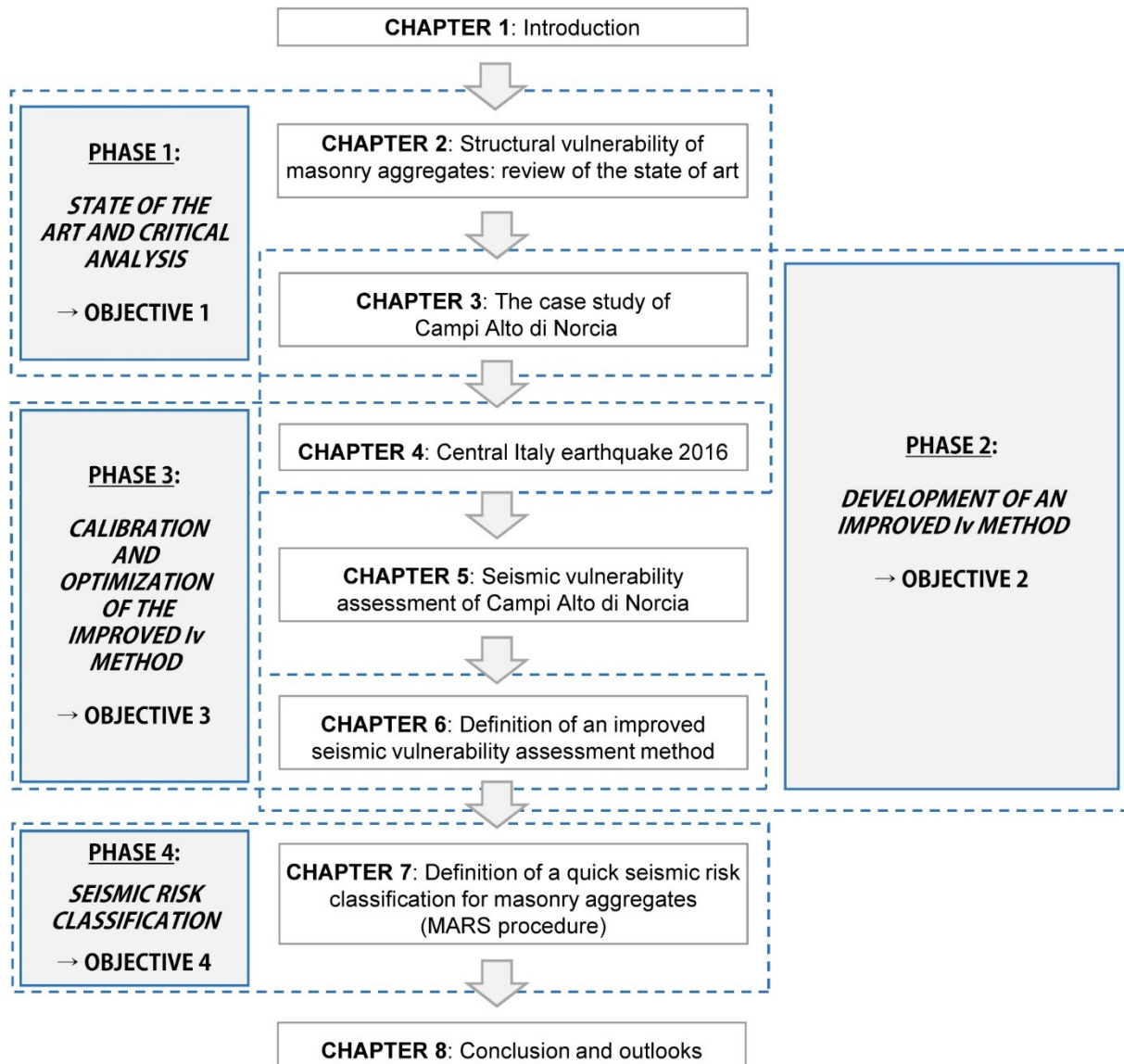


Figure 1.2: Thesis organization according to the research methodology and objectives

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CHAPTER 2. Seismic vulnerability of masonry aggregates: review of the state of art

The chapter is focused on the description of the main documents and tools used in the current scientific literature and professional activity for the analysis of masonry aggregates that form the urban environment of historical city centres and defines the main issues related to seismic risk assessment of this construction typology.

Chapter outline

2.1 Introduction

2.2 Overview of the existing methods

2.2.1 The Empirical/Expert base Methods

2.2.2 The Analytical/Mechanical Methods

2.3.3 The Hybrid Methods

2.3 Vulnerability methods used in the research activity

2.3.1 The GDNT Method

2.3.2 The Formisano Method

2.3.3 The Vicente Method

2.3.3 The Ferreira Method

2.4 Conclusions

2.5 References

2.1. Introduction

Since the early 70s, scientific research has developed a strong interest for the assessment of seismic risk of historic city centres. This led to the production of several technical documents and papers, focused on the identification of procedures for understanding the structural system, on the definition of building typologies and on the assessment of seismic vulnerability, adopting different methodologies.

During the literature review, different kind of documents are analysed, including technical standards for constructions, scientific literature, guidelines, specialized and informative books.

The current Italian technical standard, NTC (2018) and C.M. n.7 (2019), defines the masonry buildings in a historic centre, as *Structural Units* (SUs) that form *Structural Aggregates* (SAs), and provides general indications on how to analyse this type of constructions. In this sense, it recognizes that the application of assessment techniques for new buildings on masonry aggregates is unsuitable, but fails to provide resolution methods. In the context of the analysis of a historic center, it specifies that the SU shall be identified in the SA, highlighting the actions that can be derived from the adjacent structural units. Moreover, it also notes that the effects of

units' connections on the global performance of the SA, connected through a bilateral relationship, shall be accounted for.

Other technical standards, such as the Eurocode for the “Design of structures for earthquake resistance - Part 3: Strengthening and repair of buildings” (EN 1998-3:2005) and the American Standard for the “Seismic Evaluation and Retrofit of Existing Buildings” (ASCE 41/13:2014), don't provide any specific indication about aggregated buildings in historic centers.

The lack of consistent procedures and instructions in the technical standards for the analysis of this construction typology is related to the uniqueness that characterize each historic center, which doesn't allow the development of mandatory rules always suitable.

For that reasons, guidelines and specialized books are available, as result of the work achieved following different retrofits and studies carried out in old city centres, before and after seismic events (Giuffrè et al. 1993), (Milano et al. 2006), (Carocci et al. 2009), (De Sortis et al. 2009), (ReLuis 2010a), (MIBACT 2010), (Mochi and Predari 2016), (Lombardo et al. 2018).

These documents provide in detail the procedures necessary for the knowledge of building systems that form old city centres. In the knowledge process, the historic layering is a fundamental tool, because it defines the structural alterations, which brings a construction, with a specific purpose, after several events, into the current state (Figure 2.1).

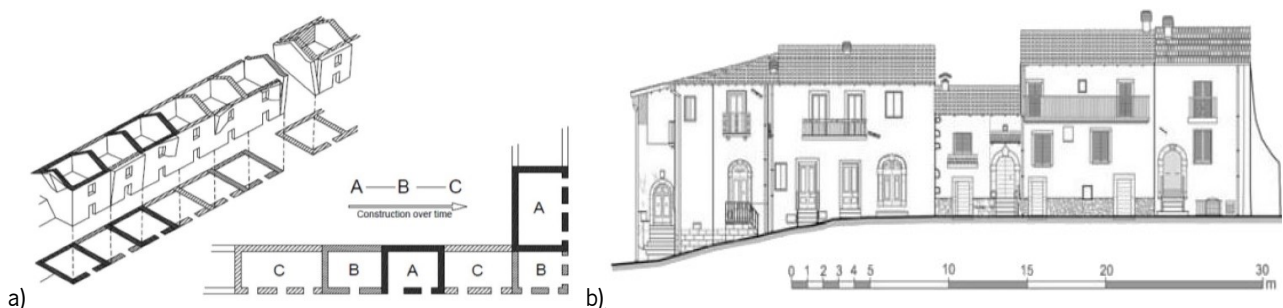


Figure 2.1: Aggregate evolution by the individuation of different SUs (adapted from Giuffrè (1990)) (a) and geometrical survey of the buildings in aggregates (b).

The frame work of an aggregate is also analysed in the urban and environmental setting in which it is formed, by the identification of the construction rules and the morphological evolution, examined in their historical context. The changes occurred over centuries leads to the superposition of different materials and construction technologies, altering the homogeneity in the original system, hence this requires the definition of hypotheses on the formation and evolution of the aggregate. The latter are then confirmed through the survey of the building geometry and the construction system, the detection of the damage and the crack pattern and the mechanical characterization of the materials.

Furthermore, these documents emphasize that the dynamic behaviour of a SA is the result of the sum of the behaviours of all the SUs in the aggregate, as observed from the damage detected in different historical centres after an earthquake (ReLuis 2010b). In this sense, the seismic response should be studied by detecting and

analysing the SUs separately from the SA (*Disaggregation Process*), to evaluate different behaviours: the activation of local mechanisms in the SU and, if they are avoided, the effective strength of the piers of the SU.

Afterwards, the SU analysed is considered inside the SA to evaluate the group effect (*Re-aggregation Process*) (Figure 2.2). It is note, that local mechanisms of SUs don't have a direct interaction with the rest of the aggregate because of the localized nature of damage, while in the global behaviour of a SU, the interactions of adjacent SUs need to be accounted for. Moreover, the principal issue for the seismic vulnerability assessment of SAs is how this interaction amongst SUs needs to be taken into consideration.

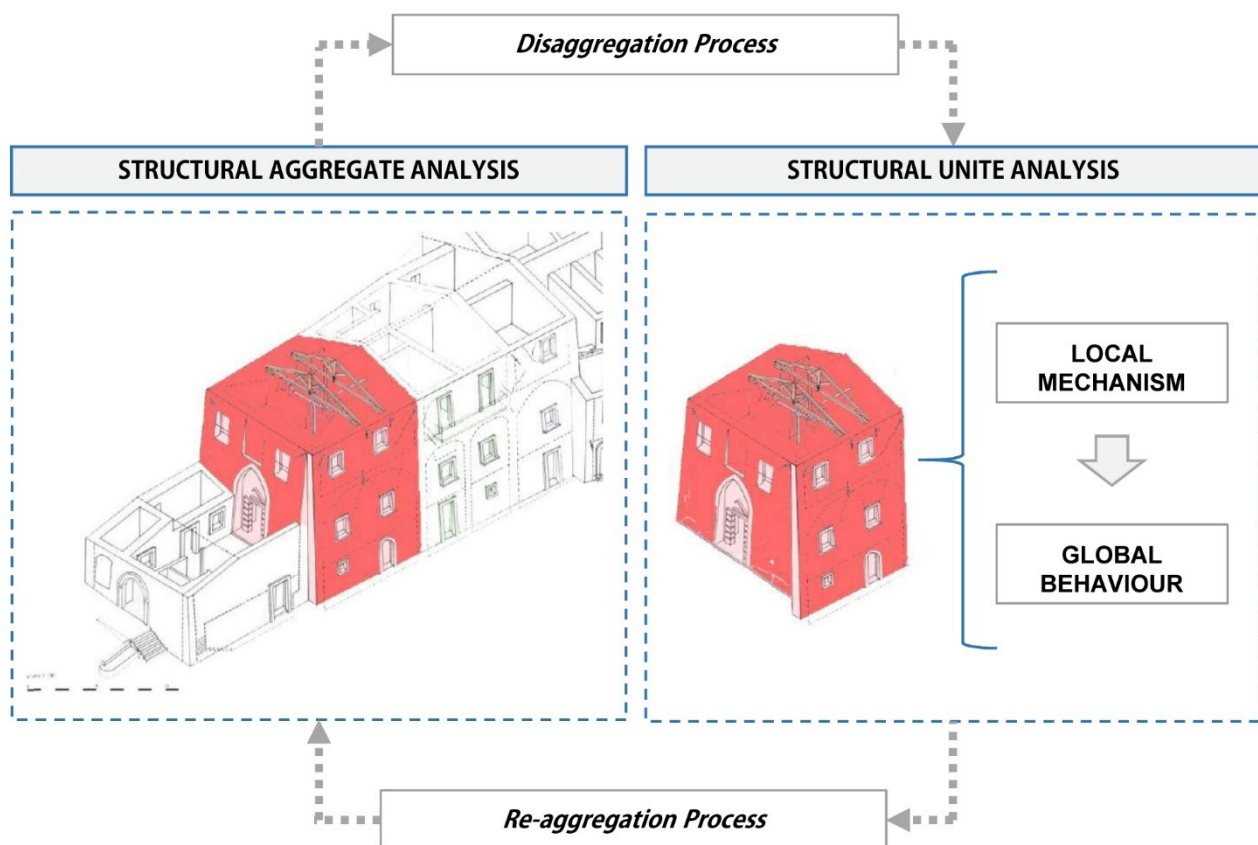


Figure 2.2: Analyses process of a SU inside an masonry aggregate to define its seismic response

In the scientific literatures, it is possible to find several approaches that provide a methodology to analyse the dynamic behaviour of SAs and to evaluate their seismic vulnerability. The results allow to develop loss model and damage scenarios of the historical centre analysed.

In the following paragraphs, the principal methods for the seismic risk assessment of SAs in old city centres, are described technically and historically, highlighting their characteristics, as well as their advantages and disadvantages.

2.2. Overview of the existing methods

The purpose of the vulnerability assessment methods is to provide the probability of a damage level for a specific building typology due to a seismic event, which is graphically represented through fragility curves. The latter are becoming more and more important as a valuable tool for the seismic risk assessment, establishing the link between the seismic hazard of a site and the effects of the predicted ground motions on the built environment. By definition, fragility curves express continuous relationships between the probabilities associated to an asset or class of assets, of reaching or exceeding predefined damage states for a range of earthquake ground motion intensities (Maio and Tsionis 2016).

Fragility curves can be developed through three main approaches, called Empirical/Expert base Methods, Analytical/Mechanical Methods and Hybrid methods. The choice to use a specific vulnerability assessment approach is a complex issue and is the consequence of the available data and the type of result that must be obtained (Figure 2.3).

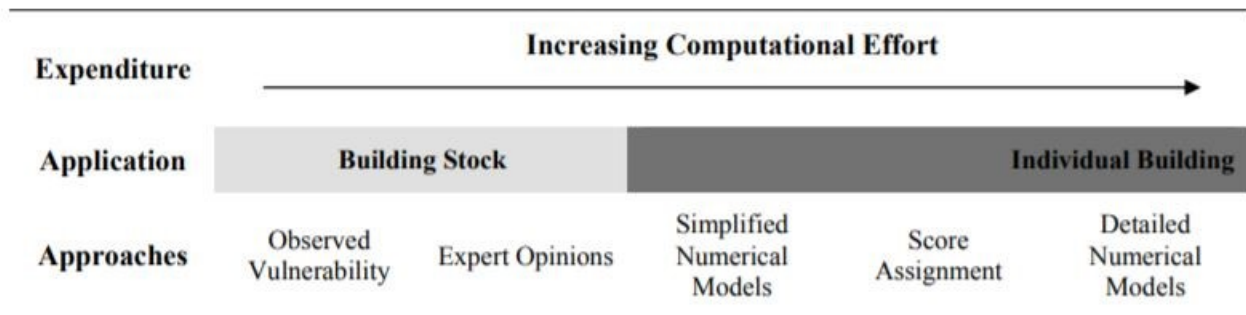


Figure 2.3: Graphic scheme about the approaches for seismic vulnerability assessment of existing buildings according to the computational effort (Lang 2002).

The Empirical/Expert base Methods, can be used for a quick and easy analysis of seismic risk at regional/urban scale. They are developed, based on the damage caused by past earthquakes and from their evaluations carried out.

A more detailed analysis' possibility consists in the adoption of Analytical/Mechanical methods to describe the structural performance of the SU and the resulting seismic vulnerability of the SA through an in-depth numerical modelling and assessment. Achieved results are characterized by higher accuracy but sophisticated modelling tools are needed to simulate the linear and nonlinear behaviour of masonry structures subjected to lateral loading (Ramos and Lourenço 2004; Lagomarsino et al. 2013), leading to a strong computational time and effort and, besides, to difficulty in the interpretation of data. Therefore, these methods cannot be used for a quick and easy analysis of seismic risk at regional/urban scale, being their application mainly limited to a specific construction typology and individual buildings in aggregates (Fajfar 1999), (Cosenza et al. 2005a), (Carocci et al. 2009), (Restrepo-Velez and G. Magenes 2004).

The Hybrid Methods try to use the potentials of the two previous methods, by evaluating the seismic response of the buildings with the help of observational methods, then supported by the mechanical concepts. The aim

of this method is to assess seismic risk at urban scale with a more sophisticated approaches than the Empirical/Expert base Methods.

Figure 2.4 shows the different characteristics, as well as the various steps necessary for the evaluation of the fragility curves with the different methods.

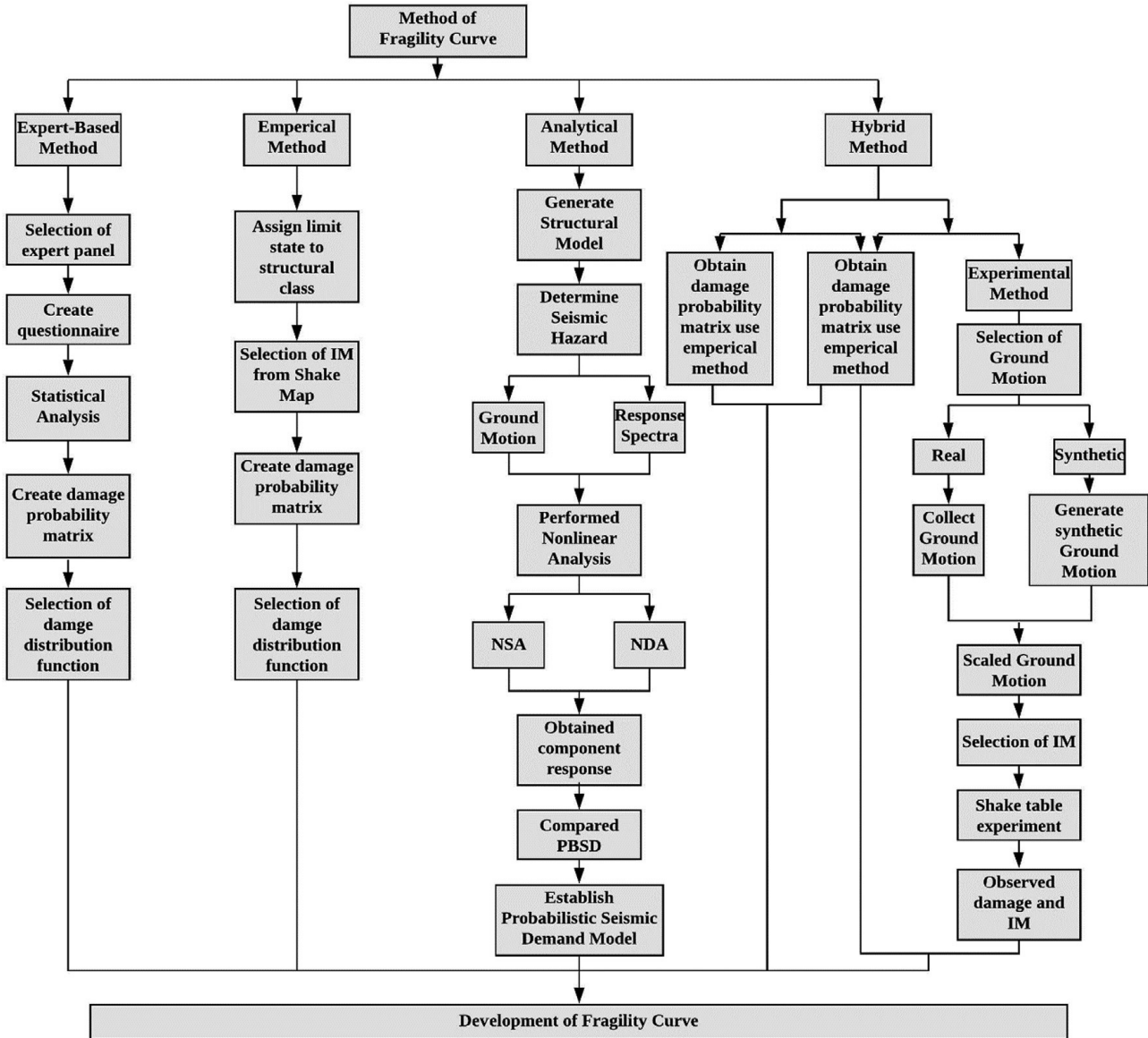


Figure 2.4: Methodologies and steps to develop fragility curves (Kassem et al. 2020).

2.2.1. The Empirical/Expert base Methods

Empirical and Expert base Methods express a single approach for the seismic vulnerability assessment, where the main difference lies in the space sample used for the development of fragility curves. While the empirical method is based on the information coming from the observed damage, the data used for the expert base method comes from skilled judgment and opinion. In both methods, the data are organized in damage probability matrices (DPMs), which describe the damage prediction of existing buildings for different seismic

intensities. The DPMs are introduced by Whitman et al. (1973) on the base of the damage observed after the 1971 San Fernando seismic event. Braga et al. (1982), after the 1980 Irpinia earthquake, develops the European version of the DPMs, adding a binomial distribution to define the damage probability for buildings with three vulnerability classes (A, B, and C), considering the MSK scale (Medvedev et al. 1964). This method defines a direct relationship between the building typology and observed damage (Corsanego and Petrini 1990), even if it doesn't take into account several construction typologies.

To overcome this issue and to fulfil an overall seismic risk assessment methodology, the RISK-UE project was established. The result is the EMS – 98 Macroseismic Scale (Grünthal 1998), that considers six different vulnerability classes (A to F), from most vulnerable to least vulnerable, for four construction typologies: (masonry, reinforced concrete, steel, and wooden). The relationship between vulnerability class and building typology is implicitly probabilistic, where the modal class is explicit most likely ("most likely vulnerability class") compared to the adjacent classes deemed probable ("Probable") and less probable ("less probable" or "exceptional"). Also the earthquake damage is reorganized, considering five damage grades (D1, D2, D3, D4, D5), from slightly damaged into fully collapsed (Figure 2.5).

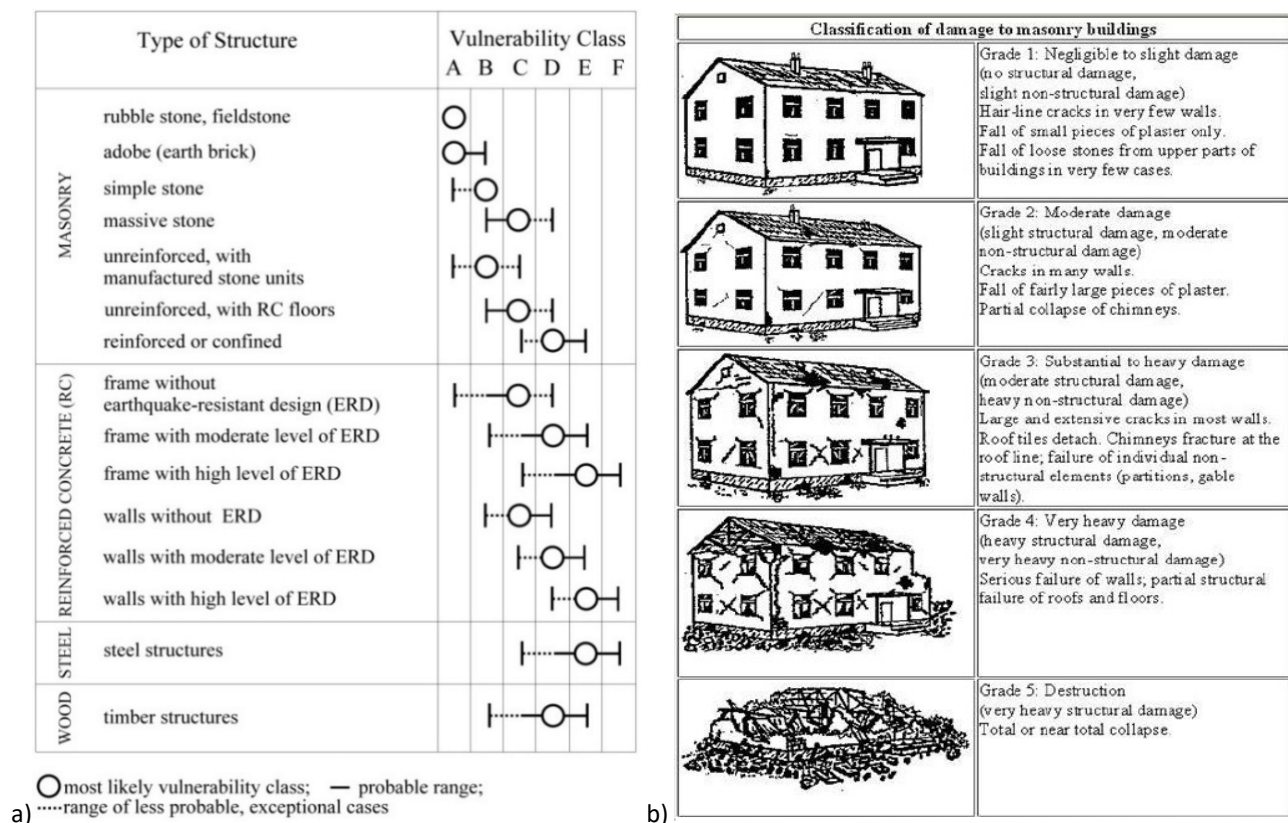


Figure 2.5: Correlation between vulnerability classes and building types according to EMS98 (a) and Damage classification of masonry buildings (b) (Grünthal 1998).

On the base of the EMS-98 Macroseismic Scale, Lagomarsino and Giovinazzi (2006) develop a hierarchical family of DPMs, where the imprecision of the correlation between intensity and damage is solved by using the fuzzy set theory (Dubois and Prade 1980), which mathematically describes the overlapping ranges of the

linguistic definitions (“few”, “many” and “most”) in terms of membership functions. In this sense, damage probability matrices (DPM) can be constructed for the different vulnerability classes defined by the EMS-98 Scale (Ortega 2018). Figure 2.6b shows, for example, that few buildings with vulnerability class C suffer the damage grade 1 for an earthquake with a macroseismic intensity of VI, just for the same vulnerability class most buildings suffer a damage grade 5 for a macroseismic intensity of XII. The parameterization of the cumulative DPMs leads to the formulation of an analytical relationship, [2.1] and [2.2], between the seismic vulnerability (ranging from 0 to 1), the macroseismic intensity I (from V to VII) and the average value of the expected damage distribution μ_D (from 1 to 5). The factor “ Q ” represents the ductility index, which takes into account the ductility of a determined construction typology, typically ranging from 1 to 4.

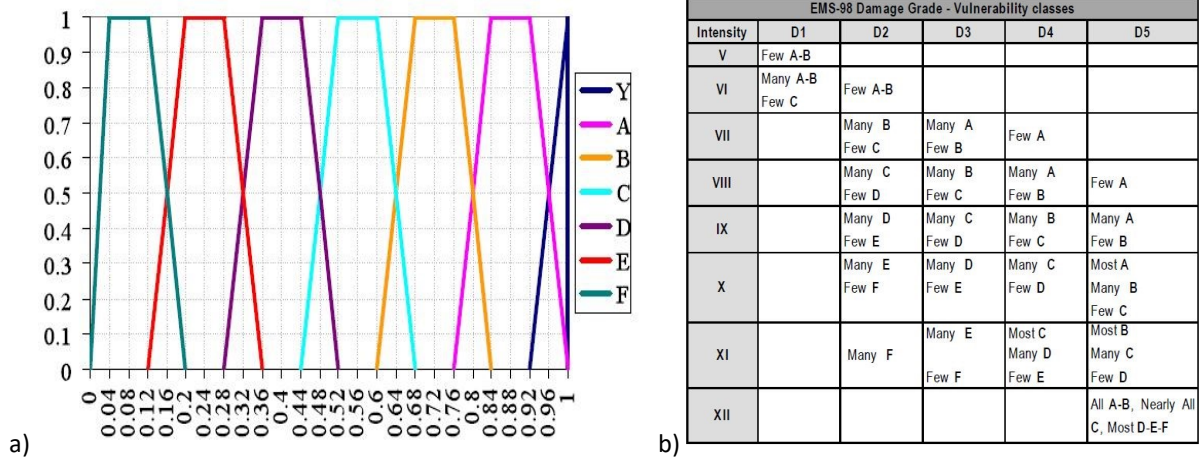


Figure 2.6: Fuzzy sets (Bernardini et al. 2010) (a) and DPM of the vulnerability classes, according to the EMS98 – Scale (Boschi 2015) (b)

$$\mu_D = 2.5 + 3 \cdot \tanh\left(\frac{I + 6.25 \cdot V - 12.7}{Q}\right) \cdot f(V, I), \text{ for } 0 \leq \mu_T \leq 5 \quad [2.1]$$

$$f(V, I) = \begin{cases} e^{\frac{V}{2}(I-7)} & I \leq 7 \\ 1 & I > 7 \end{cases} \quad [2.2]$$

Besides the macroseismic approach, the vulnerability index method is another empirical method developed in the last years of the '80s by Guagenti and Petrini (1989) and later revisited by (Bernardini et al. 2007), (Barbat et al. 2008) and (Vicente et al. 2011a). The method is based on defining the seismic vulnerability index (I_v) of a building through the analysis of several parameters. The latter are characterized by four different vulnerability classes - from A (optimal) to D (unfavourable) - and by a specific weight that takes into account its relative importance in the dynamic behaviour of the building. Finally, the I_v evaluation is performed, after the comprehensive survey of the building, by the weighted sum of the vulnerability class of each parameter [2.3]. Resulting values are usually normalized in the range from 0 (very low vulnerability) and 100 (very high vulnerability) (Cherubini et al. 1999) [2.4].

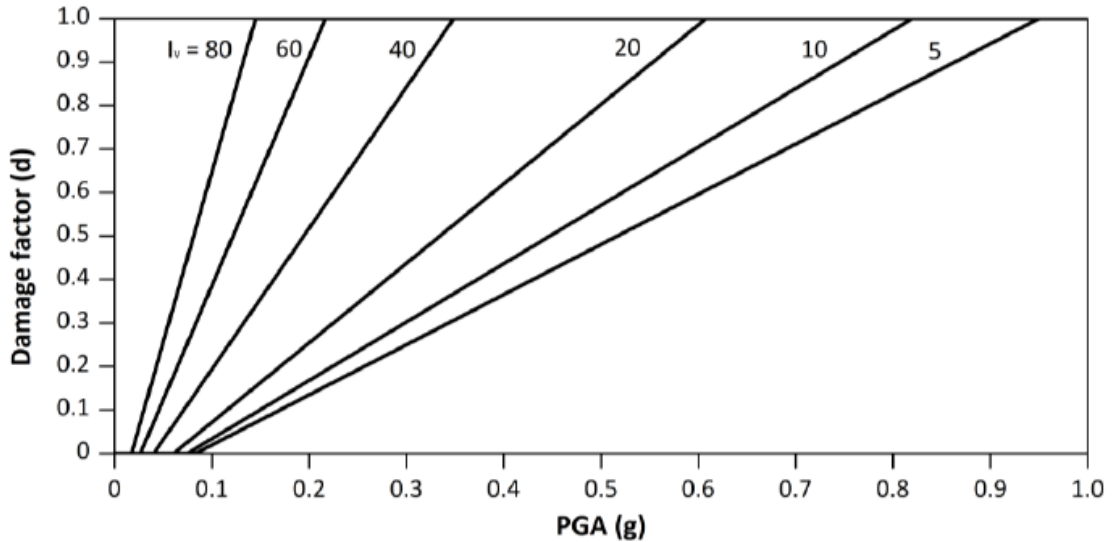
$$I_V^* = \sum p_i \cdot c_i \quad \text{where} \begin{cases} c_i = \text{vulnerability class "i"} \\ p_i = \text{weight i} \end{cases} \quad [2.3]$$

$$I_V = I_V^* \text{ normalized to 100\%} \quad [2.4]$$

The post-earthquake damage observation is used to calibrate the I_V with a global damage factor (d) for a building typology, subjected to a specific macroseismic intensity or PGA. The damage factor, between 0 and 1, defines the ratio of repair and replacement cost and increase linearly between two PGA threshold values. Figure 2.7 shows the vulnerability index formulation of the Italian National Group of Defense from Earthquakes (GDNT 1994) and vulnerability functions calibrated after data from the 1976 Friuli and 1984 Abruzzo earthquakes.

Parameter	Class (C_{vi})				Weight (p_i)	Vulnerability index
	A	B	C	D		
P1. Type and organization of resisting system	0	5	20	45	1.00	$I_V = \sum_{i=1}^{11} C_{vi} \times p_i$
P2. Quality of resisting system	0	5	25	45	0.25	
P3. Conventional resistance	0	5	25	45	1.50	
P4. Building position and type of foundations	0	5	25	45	0.75	
P5. Type of horizontal structural system	0	5	15	45	1.00	$0 \leq I_V \leq 382.5$
P6. Plan configuration	0	5	25	45	0.50	
P7. Elevation configuration	0	5	25	45	1.00	Normalized index $0 \leq I_V \leq 100$
P8. Maximum distance between walls	0	5	25	45	0.25	
P9. Type of roofing structural system	0	15	25	45	1.00	
P10. Non-structural elements	0	0	25	45	0.25	
P11. Conservation state	0	5	25	45	1.00	

a)



b)

Figure 2.7: Original formulation of “The Italian National Group of Defense from Earthquakes (GDNT 1994) (a) and Vulnerability functions to relate damage factor (d) and peak ground acceleration (PGA) for different values of I_V (Guagenti and Petrini 1989) (b).

The macroseismic and the vulnerability index method defines fragility curves with different parameters: while the first one provides the variation of the mean damage grade (μ_D) in relation to the EMS-98 seismic intensity, in the I. method the damage factor (d) is influenced by the PGA. The Federal Emergency Management Agency (FEMA) developed a methodology (HAZUS MH 1999) to transform the economic damage (d) in terms of structural damage (μ_D) with an analytical formulation [2.5], considering a probabilistic damage distribution, derived from the discretisation of a beta distribution:

$$\mu_D = 4 \cdot d_e^{0.45} \quad [2.5]$$

Other authors establish a relationship between d_e and μ_D (Figure 2.8), such as Rojahn et al. (1985), Bramerini et al. (1995) and Dolce et al. (2000), where the economic index related to specific damage grades are different.

Damage grade, D_k	0	1	2	3	4	5
Level of damage	No damage	Slight	Moderate	Severe	Very severe	Destruction
Economic damage index, d_e						
ATC-13 (1985)	0.000	0.050	0.200	0.550	0.900	1.000
Bramerini et al. (1995)	0.000	0.010	0.100	0.350	0.750	1.000
HAZUS (1999)	0.000	0.020	0.100	0.500	1.000	1.000
Dolce et al. (2000)	0.000	0.035	0.145	0.305	0.800	1.000

Figure 2.8: Correlation between damage grade and economic damage index (Vicente et al. 2011b)

Gathering from the formulation [2.5], the correlation between the vulnerability index (I_v) and the vulnerability parameter (V) of the equation [2.1] is developed, by converting the PGA into the EMS-98 seismic intensity scale. Firstly, the values of the MCS scale are transformed in the EMS-98 seismic intensity through the analytical expression provided by (Margottini C., 1992):

$$I_{EMS-98} = 0.734 + 0.814 \cdot I_{MCS} \quad [2.6]$$

Following this, a logarithmic relationship between PGA and the MCS (I_{MCS}) is used (Guagenti and Petrini 1989).

$$\ln(y) = a \times I_{MCS} - b \quad \text{with } a = 0.602 \text{ and } b = 7.073 \quad [2.7]$$

Finally, the vulnerability curves can be compared and a final relationship between the vulnerability indexes is defined (Figure 2.9). In the scientific literature there are different formulations, such as (Vicente et al. 2011b), (Vicente et al. 2014) (Maio et al. 2015), (Basaglia et al. 2016), (Ferreira et al. 2017), (Athmani et al. 2018),

(Ortega et al. 2018) and (Mosoarca et al. 2019), that describe the correlation between the I_v and the vulnerability index V , according to the construction typology analysed.

V and I_v correlation expressions	References
$V = 0.56 + 0.0064 \times I_v$	Vicente et al. (2011)
$V = 0.58 + 0.0064 \times I_v$	Maio et al. (2015)
$V = 0.592 + 0.0057 \times I_v$	Ferreira et al. (2017)
$V = 0.46 + 0.012 \times I_v$	Ortega et al. (2018)
$V = 0.46 + 0.0056 \times I_v$	Mosoarca et al. (2019)

Figure 2.9: Correlations between V and I_v indices used by researchers (Kassem et al. 2020).

These methods are applied by several authors for the seismic risk assessment of different old city centres in Mediterranean countries, like for example in Italy with Cavaleri et al. (2017) and Romis et al. (2020), in Morocco (Cherif et al. 2017), Algeria with Kherroubi et al. (2009) and Mourabit et al. (2014), Spain (Lantada et al. 2010) and Portugal with Ferreira et al. (2017) and Maio et al. (2019).

The principal limitations of the Empirical Methods are their high dependence on the building characteristics and on the seismic scenario(s) originally used to calibrate and validate them, these aspects influence directly the individual parameters of considered formulations, which, in turn, may lead to low accuracy of vulnerability and damage results. Moreover, the lack of information related to the construction typology analysed, as well as the possible seismic intensity degrees, can lead to a rough estimation of damage scenarios, caused by future earthquakes (D'Ayala and Novelli 2010). Another limitation is related to the definition of a single value for the vulnerability, which can potentially increase the difference between the real and expected damage.

However, when they are applied to regions with similar characteristics, a realistic indication of the expected damage should result, because many uncertainties of these methods are inherently accounted for (Calvi et al. 2006). Furthermore, the empirical methods are quick to apply, allowing to provide a preliminary screening of territorial criticisms, that can be later analysed in detail.

2.2.2. The Analytical/Mechanical Methods

In the case of Analytical/Mechanical Methods the DPMs are developed through the analysis of quantitative models that try to foresee the structural behavioral of a construction typology, contrary to what happens with empirical methods, in which the observed damage is used to predict future earthquakes. The evaluation of the building response during a seismic event, requires, first of all, the characterization of construction technologies and quantification of the potential ground motions. Afterwards, numerical models are involved, performing different types of analysis, such as linear/nonlinear static or dynamic analysis, in relation to different limit states and PGAs. Finally, the data is treated statistically to evaluate the probability of different damage, validating and comparing, finally, the results with the real damage observed after earthquakes or experimental data.

Singhal and Kiremidjian (1998) are among the first, who developed fragility curves on the base of DPMs, derived from numerical models of reinforced concrete buildings. In this case, the structural damage is determined through nonlinear dynamic analysis, using different PGAs and DPMs are validated through the damage detected after the 1994 Northridge earthquake.

In Europe, the analytical approach is used by Masi (2003) to characterize the seismic behaviour of reinforced concrete frames, that form primary structural typology of Italian buildings constructed after the '60s, featuring their vulnerability with the EMS-98 Scale. In the same way, Rossetto and Elnashai (2005) consider several European RC structures, as sample space.

With the improvement of structural analysis and design software, nonlinear analyses are used to better describe the seismic demands of different types of structures, categorized, generally, into two groups: Non-Linear Time History Analysis (NLTHA), and Non-Linear Static or Pushover Analysis (NLSA/POA). Several studies are performed in this manner, such as (Fragiadakis and Vamvatsikos 2010), (Zacharenaki et al. 2013), (Zameeruddin and Sangle 2016), (Liu and Kuang 2017) and (Rahmani et al. 2018).

The principal advantage of this method is the accuracy of the results, since it is possible to consider all type of uncertainties. On the other hand, the production of fragility curves is extremely computationally intensive and time consuming, especially for SAs in historical centres, where modelling guidelines are missing (AIMS 2019). Moreover, the reliability of the results decrease for areas with varied construction typologies, built with different materials.

2.2.3. The Hybrid Methods

The Hybrid Method develops DPMs and fragility curves through the combination of the observed damage with the expected one, defined with mechanical models of the building typology analysed. In this sense, it is possible to consider more types of damage and seismic intensities, compared to those obtainable from territorial surveys after an earthquake. The seismic risk of an urban centre is evaluated through a risk index, as a ratio among the structural capacity and the seismic demand. The purpose of this method is, therefore, to sustain, rather than to replace, the empirical DPMs and fragility curves based on the observational damage.

An example of hybrid methods is Barbat et al. (1996), who compares and calibrates vulnerability curves, derived from the vulnerability assessment of a Spanish building stock, performed with empirical methods, and the capacity/demand ratios of models with similar characteristics of the construction typology analysed.

Moreover, Kappos et al. (2006) combines statistically the results obtained by the nonlinear dynamic analysis of typical Greek building models, with the damage observed after the 1978 earthquake in Thessaloniki.

The main issue of these studies is related to the calibration of two curves, obtained from different sources: observed damage and analytical analysis. While the latter can take into account the uncertainties of the structural behaviour, the variability of empirical data are not quantifiable. Furthermore, the criticisms, highlighted in the previous methods for evaluating the seismic risk in historic centres, such as the difficulty of modelling of SAs and the high dependence on the building characteristics, continue to persist.

Other type of methods, called VULNUS (Bernardini et al. 1990) and FAMIVE (D'Ayala and Speranza 2003), can be considered hybrids, since these combine the characteristics of the vulnerability index methodology with mechanical evaluations of the structures. In this sense, the application of these tools is suitable also for the vulnerability assessment of masonry aggregates in historical centres.

VULNUS, is a procedures based on the limit analysis of structural macroelements for the evaluation of the probability of collapse (V_u or V_g), assumed by reaching a damage higher than the D4 level according to the EMS - 98 Scale. Through the survey of the building, it is necessary to evaluate four different Indexes [2.8]:

$$V_u \text{ or } V_g = f(I_1, I_2, A, a), \text{ with } \begin{cases} I_1 = \frac{\min(V_x; V_y)}{W} \\ I_2 = \min(I'_2 + I''_2)_i \\ I_3 = \sum_i \frac{(p_i \cdot c_i)}{45 \cdot 3.15} \rightarrow a \end{cases} \quad [2.8]$$

Where the first Index (I_1) takes into account the shear failure at the ground floor of the building and is defined as the ratio of the in-plane shear strength (V) of the walls in the x – and y - directions to the total weight (W) of the building. I_2 is the collapse multiplier for the out-of-plane behaviour, considering the flexural strength of the most critical external wall, by summing the resistance of the piers (I'_2) and spandrels (I''_2). I_3 corresponds to the I_1 assessed with the GDNT (1994) method and allows to evaluate the uncertainty factor (a) with the fuzzy-set theory (Figure 2.10a). Finally, “ A ” represents the ratio between the average value (on the building masses) of the absolute acceleration response to seismic excitation (PGA/g). Taking into account the judgments expressed in the survey of the building and the quality of the information collected, the parameters I_1 , I_2 and I_3 are evaluated as fuzzy sets with some degree of membership. The latter is calculated within the interval $[0,1]$, which measure the probability of collapse for a building (V_u) or for a group of buildings (V_g). From these analyses, lower bounds (E_{Low}), upper bounds (E_{upp}) and central values (E_{white}) of the cumulative probability functions, that describe the expected damage, can be calculated (Figure 2.10b).

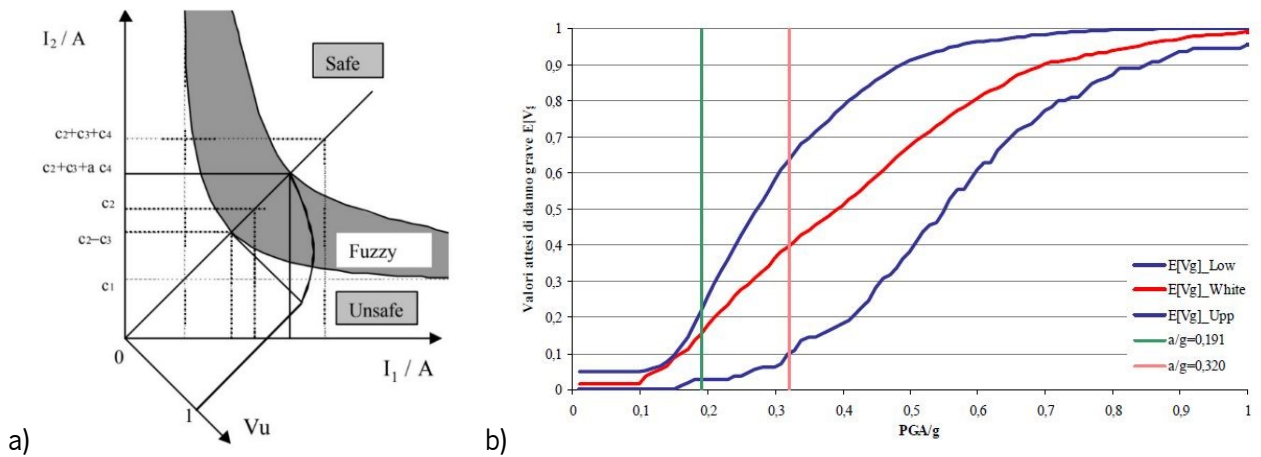


Figure 2.10: Seismic vulnerability model of masonry buildings assumed in VULNUS, where the probability of collapse V_u measures the membership to the “Unsafe” zone (Bernardini 2001) (a) and fragility curves obtained through the VULNUS procedure, considering the building stock of the old city centre of Campi Alto di Norcia, subjected by the PGA evaluated with the maximum macroseismic intensity, historically measured in the area, ($pga/g = 0.191$) and with the old Italian Code for constructions (NTC 2008) ($pga/g = 0.191$) (Valluzzi et al. 2009) (b).

The Failure Mechanism Identification and Vulnerability Evaluation (FaMIVE) is based on the definition of the most probable collapse mechanism of building façades, considering their in-plane and out-of-plane failures (Figure 2.11). The collapse multipliers are evaluated with the aid of survey sheets, in which a range of possible collapse configurations of each façade wall of the building are assumed. In this sense the geometrical features, the loads, the restraints and boundary condition of the structural macroelement are all taken into account. The data is post processed to define degrading capacity curves through a non-linear pseudo-static structural analysis. The mechanism with the lowest seismic capacity is then identified. By combining these results with the ground motion data of possible seismic scenarios it is possible to define fragility curves and performance points of the building façades in terms of intermediate and ultimate displacements or ultimate acceleration.

These procedure are applied to several case studies of old city centres, (D'Ayala and Speranza 2004), (Valluzzi et al. 2009), (Munari et al. 2009), (Formisano et al. 2013), (Novelli et al. 2015), where the information relates only to external onsite inspections. The principal disadvantage is the high level of uncertainty concerning the geometric composition, the material properties, the boundary conditions and loads involved among the orthogonal walls of the construction analysed, given the randomness of its structural features. In this context, Cosenza et al. (2005b) tries to solve these kind of uncertainties with similar procedures but regarding RC buildings, which are more standardised.

Moreover, methods like VULNUS and FaMIVE don't provide clear indications on how to match the structural capacity with the seismic demand to evaluate the probability of exceeding specific limit states or damage and, therefore, loss assessments are limited.

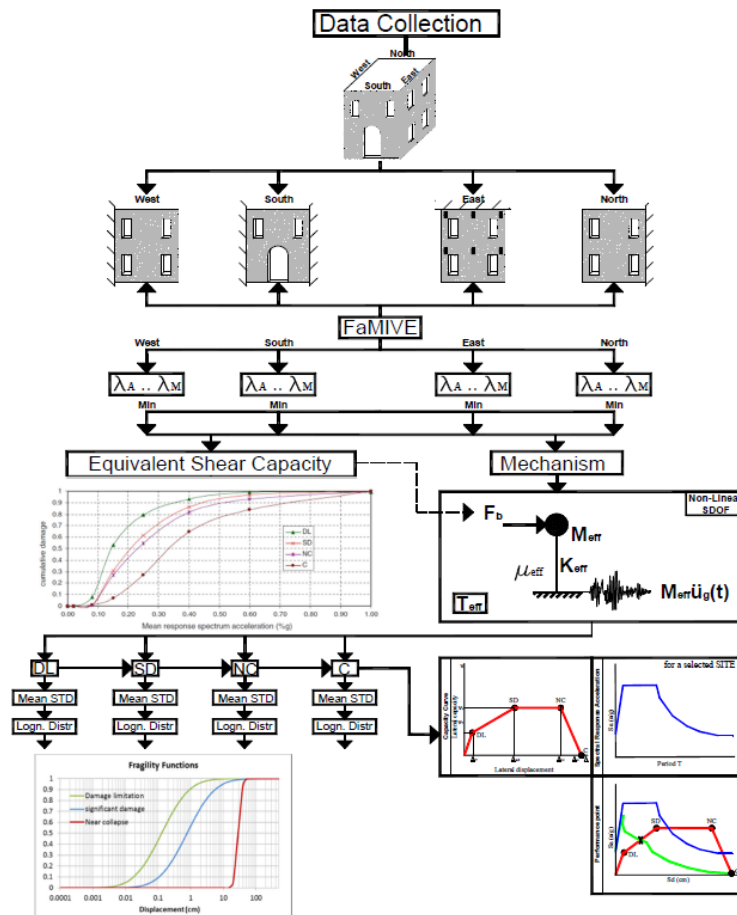


Figure 2.11: Workflow of the FaMIVE method for derivation of fragility functions (D'Ayala et al. 2015).

2.3. Vulnerability methods used in the research activity

The review of literature of the vulnerability methods provides a clear knowledge of the tools and data necessary for their application in the context of historic centres. Moreover, advantages and disadvantages are highlighted for assessing the seismic risk of masonry aggregates, the principal construction typology that form old city centres. The accuracy of the results of these methods is directly linked to the reliability of the fragility curves that are developed. Here, the parameters required for the evaluation of fragility curves are analysed and compared between the methods in the following Table 2. 1.

Table 2. 1: Knowledge of the parameters required for the evaluation of fragility curves of historical centres

Parameters required for the fragility curves development in historical centres.	Empirical Methods	Analytical Methods	Hybrid Methods
General			
Geographical applicability	✓		✓
Surrounding environment (streets, ...) is evaluated	✓		
High training Level of the Technician		✓	✓
Seismic Capacity			
Understanding the structural System	✓	✓	✓
Evaluation of In – plane Mechanisms	✓	✓	✓
Evaluation of Out of plane Mechanisms		✓	✓
Analyses of the Material properties used in construction		✓	
Knowledge of the Structural detailing		✓	✓
Survey of Geometrical irregularities		✓	
Study of the Construction Age	✓		
Vulnerability of non-structural elements	✓		
Seismic Demand			
Seismic Hazard knowledge	✓	✓	✓
Evaluation of a database of Observed damage from prior earthquakes	✓		
Knowledge of Site specifics		✓	✓

Since the objective of the present research activity is the development of a procedure for the seismic risk estimation on urban scale of masonry aggregates in historical city centres, empirical methods are used, trying to overcome their principal limits. Four traditional approaches of the vulnerability index methodology are selected for the following phases of the research activity: the GDNT (1994), the Formisano et al. (2009), the Vicente (2010) and the Ferreira et al. (2012) methods, being more appropriate for the considered context.

As previous said, the seismic vulnerability of a building is assessed by the definition of a vulnerability index through the weighted sum of different parameters that consider the dynamic behaviour of the structure [2.3],

[2.4]. Each method has a specific survey sheet, in which it is possible to assign a score and a weight to each parameter, according to precise rules.

The methods selected differs for number of parameters used and construction typology adopted for their calibration. In this sense, the Vicente (2010) and Ferreira et al. (2012) method are developed for structures built in historical centres of the Portuguese town, while GDNT (1994) and Formisano et al. (2009) method are applied for buildings, located in the Italian territory. Furthermore, the GDNT (1994) form is focused on isolated buildings, while the Formisano et al. (2009) and Vicente (2010) methods assess the seismic vulnerability of SUs in SAs, considering the aggregate effect with specific parameters. Ferreira et al. (2012) method represents a different approach than the others, as it considers the aggregate in its entirety and, therefore, can provide a preliminary screening of the vulnerabilities in the historical centre analysed.

The following paragraphs show the different parameters of each method in detail, to provide a comprehensive understanding of the analyses performed and results achieved in the next chapters.

2.3.1. The GDNT method

The GDNT form is among the first methods developed and represents a reference and a basis on which all the others are developed. It adopts for the I_v evaluation eleven parameters (namely P1 to P11) and four vulnerability classes (A to D, being A the best condition and D the worst one); the method was developed for isolated buildings, taking into account their geometry, structural and non-structural features, the floors and roofing typologies, walls thickness and conservation state. The maximum value of the achievable non-normalized I_v [2.4] is equal to 382.5.

PARAMETER 1: Organization of Vertical Structures

This parameter evaluates the structural features that guarantee the “Box Behaviour” of the structure, such as the connections between the orthogonal walls, the presence of metal ties against the thrusts of arches and roofs and the presence of concrete rings at each floor to connect the slabs with the masonry piers. In this context, the knowledge of the retrofits systems plays a fundamental role, as well as the history of the transformations of the structure (Figure 2.12). Class A is assigned for SUs built according to the recent technical codes for constructions or for retrofitted buildings, that consider the prescriptions of the most recent repair rules. For class B, the connections are guaranteed by concrete rings on the perimeter of the building and by metal ties on each floor. If the building has no metal ties on each floor or the orthogonal walls are not connected effectively, class C or D should be applied.

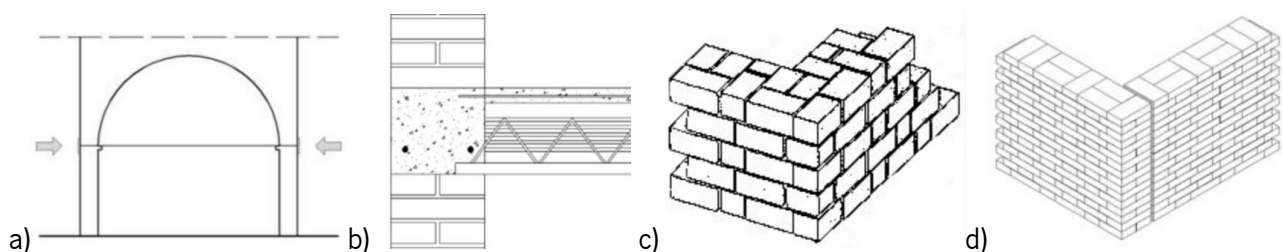


Figure 2.12: Structural details for the assignment of score A (a), B (b), C (c) and D (d) for the parameter 1.

PARAMETER 2: Nature of vertical structures

This parameter evaluates the quality of the resisting system, towards the type of material and the conservation state of the masonry (Figure 2.13). The mechanical characteristics of the materials are taken into account, considering the quality of the blocks and mortar, the regularity and disposition of stones or bricks, as well as their homogeneity, and the thickness of the mortar joints. The presence of elements (diatones) across the two faces of the wall is also evaluated. Class A identifies good quality brick/stone/tuff masonry walls, accompanied with connections between the two faces and with a homogeneous distribution throughout the structural system. For class B the masonry walls are well squared but the distribution is not homogeneous, while for class C the masonry disposition is not regular and without connections between the two faces of the wall. Finally, class D defines a very bad masonry quality, both for bricks, stones and tuff.

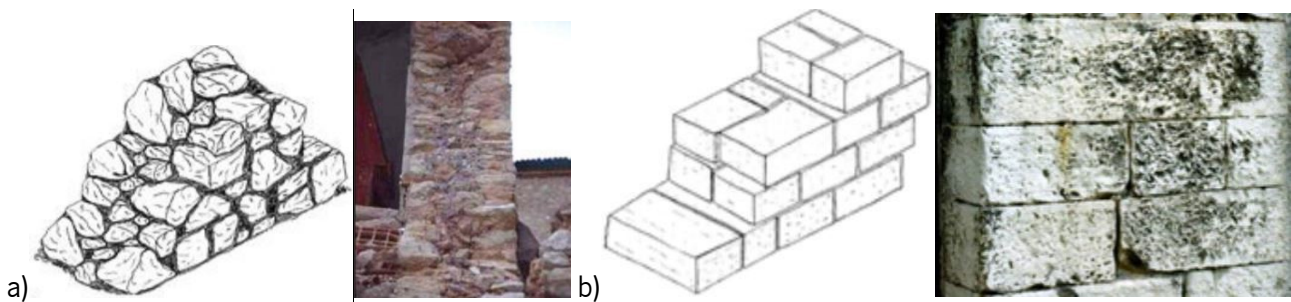


Figure 2.13: Nature of vertical structures according to the masonry quality evaluation sheets (Borri and De Maria 2019).

PARAMETER 3: Qualitative resistance

Under the hypothesis of global “box-behaviour”, this parameter provides an estimation of the resistance of a masonry building to the horizontal actions, through a simplified formulation that compares the building, in its weaker direction, to an equivalent shear wall (C). The method itself notes, that the reliability of the results decreases for buildings built before 1900, which are not retrofitted, and where the global “box-behaviour” is not guaranteed. The results achieved must, therefore, should be adopted with extreme caution. The shear resistance is evaluated through the formulation of Turnšek and Cacovic (1971), as a function of the characteristic tangential stress of the masonry wall (τ_k) and the average vertical compressive load σ_0 , which bears on it. For its evaluation it is, therefore, necessary to define for each plan:

- The number of floors starting from the ground floor (N);
- The average covered area over the section that identify the verification plan (A_i);
- The total cross sectional areas of resistance elements in the two orthogonal directions (A_x , A_y);
- The characteristic tangential shear resistance of the masonry type of the building (τ_k);
- The average height of each storey (h);
- The average weight, per unit of covered area, of a building level, as the sum of the weight of a slab and a wall within a floor (q);

The average weight (q) can be evaluated, in relation to the average specific weight of the masonry (p_m), the average weight per unit area of the floor (p_s) and the average of the floor height (h), with the following expression [2.9]:

$$q = \frac{(A + B)h}{A_t} \cdot p_m + p_s \quad [2.9]$$

Finally, the average shear resistance is evaluated with formulation [2.10]:

$$C = \frac{a_o \tau_k}{qN} \sqrt{1 + \frac{qN}{1.5 a_o \tau_k (1 + \gamma)}} \quad [2.10]$$

Where a_o is the ratio between the minimum of the cross sections ($A = \min [A_x, A_y]$) and A_t , while γ is the ratio among the maximum cross sections ($B = \max [A_x, A_y]$) and A .

The attribution of a building to one of the four classes takes place on the basis of the ratio $\alpha = C/\bar{C}$ between the value of C , obtained as mentioned above, and the value of $\bar{C}$ assumed equal to 0.4. For class A, the parameter α is above 1.0, while for class B and C is in the range of 0.6 - 1 and 0.4 - 0.6. Finally, class D identifies buildings with an α under 0.4.

PARAMETER 4: Location of building and type of foundation

It assesses the location of the building, in relation to the morphology of the surrounding area, and estimates the foundations characteristics, according to the soil typology, the natural slope and the possible presence of different levels (Figure 2.14).

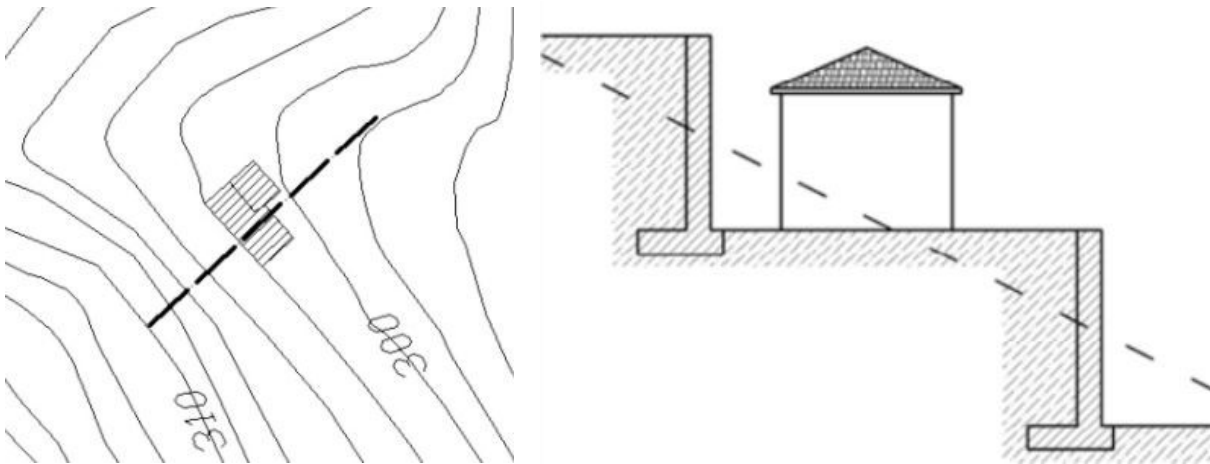


Figure 2.14: Analysis of the average natural elevations on which the building is constructed.

PARAMETER 5: Floor typology

It defines the role of the floors in relation to the global “box-behaviour” of the building, considering the connections to the vertical walls, the stiffness and the orientation of the slabs. Several schemes (Figure 2.15 and Figure 2.16) help to identify the different floor typologies. Class A is related to rigid and well connected slabs without staggered floors. In case of staggered floors, class B has to be assigned. For deformable slabs, but well connected, with or without staggered floors, the class C is selected. Finally, deformable/rigid slab and bad connected are identified with class D.

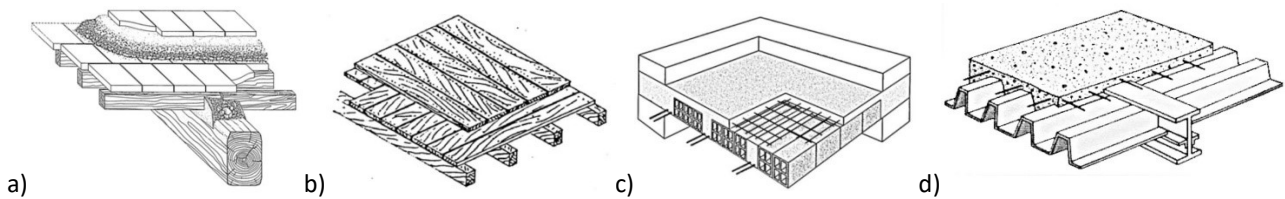


Figure 2.15: Deformable floors: Wooden floor with bricks (a); Semi-Rigid: wooden floor with double crossed planking (b); Rigid floors: concrete slab (c) and steel slab (d).

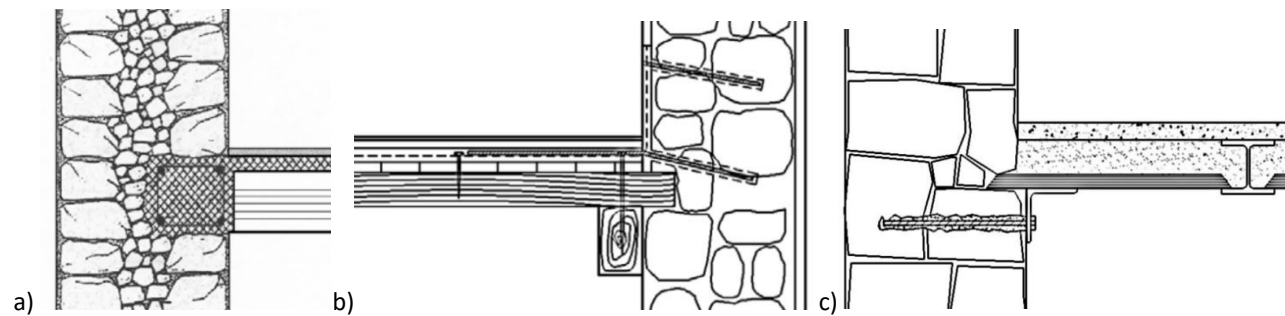


Figure 2.16: Bad (a) and good (b, c) connection to the vertical elements

PARAMETER 6: Plan regularity

The parameter analyses the plan regularity of the building (Figure 2.17), by the calculation of two coefficients (β_1 and β_2) to define its class. In case of rectangular buildings, $\beta_1 = a/l$ defines the relationship between the shorter (a) and the longer side (l). In case of other regular plan form, it is necessary to take into account the measures of their deviations (b), with the variable $\beta_2 = b/l$.

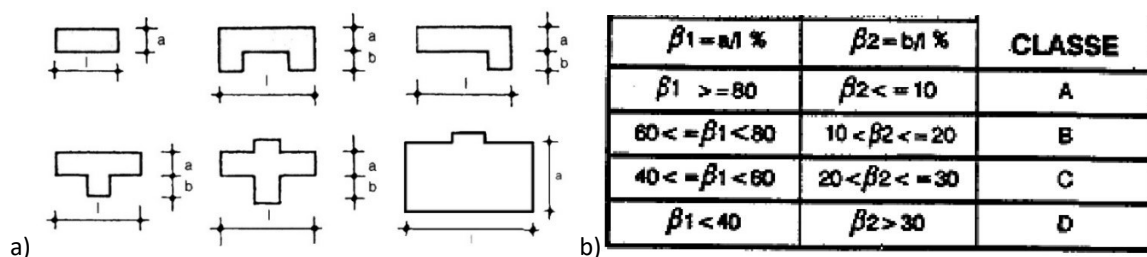


Figure 2.17: Scheme of the possible plan configurations (a) and sheet for the evaluation of parameter 6, according to the GDNT form (b).

PARAMETER 7: Height regularity

This parameter considers the presence of different stiffness at each level of the structure, considering the presence of porticos or lodges at the ground level or the existence of towers or turrets (Figure 2.18). The score is assigned by measuring the difference of masses at each level ($\pm \Delta M/M$), with a permissible deviation of upto 10% between consecutive floors. The ratio $\pm \Delta M/M$ can be replaced with the ratio $\Delta A/A$, which defines the covered area of the plan and its variation in different levels, considering the masonry type is the same at each level of the structure.

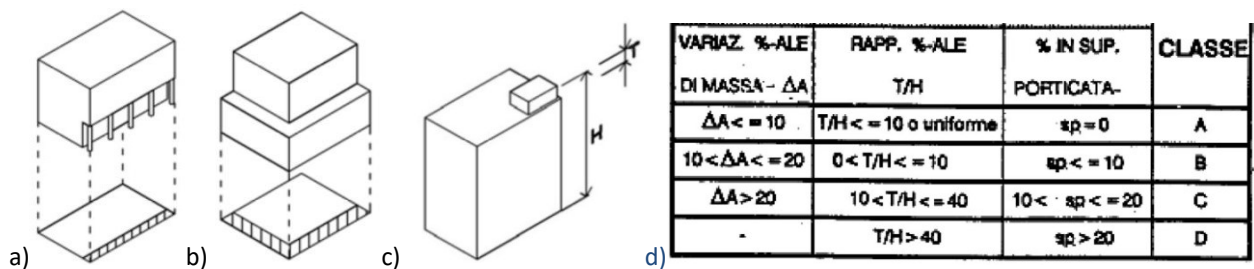


Figure 2.18: Scheme for the evaluation of the porticos dimensions (a), the plan difference of two consecutive floors (b), the presence of towers or turrets (c) and sheet for the evaluation of parameter 7, according to the GDNT form (d).

PARAMETER 8: Distribution of plan resisting elements

This parameter takes into account the presence of structural walls (excluding partitions) crossed by transverse walls, which constitute an effective constraint for it (Figure 2.19). The vulnerability classes are defined according to the most unfavourable ratio between the distance of the transverse walls (l) and the thickness (s) of the wall considered. Class A identifies the l/s ratio below 15, while class B and C is between 15 - 18 and 18 - 25. Finally, class D applies for a ratio greater than 25.

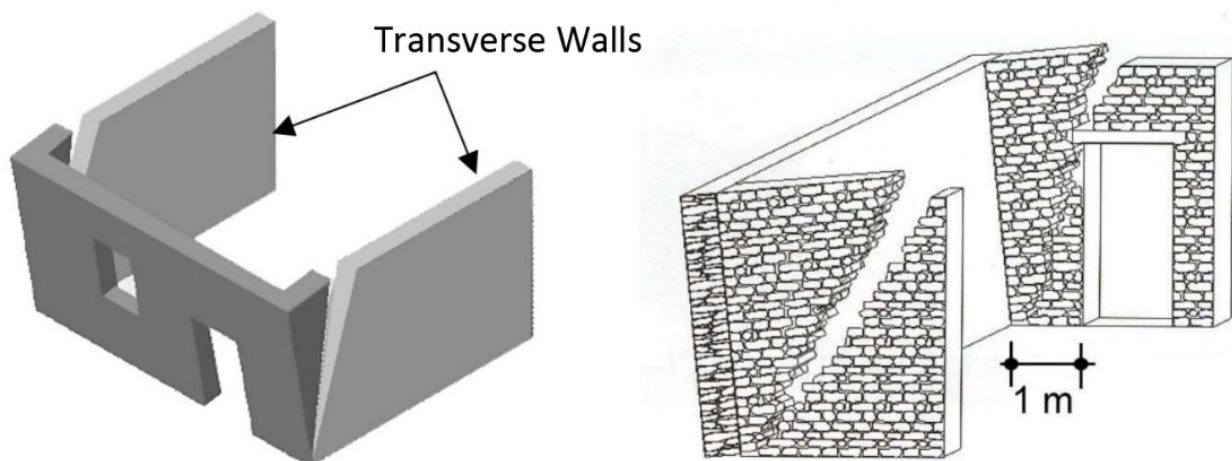


Figure 2.19: Scheme of the structural behaviour described by the parameter, that is the not effective constraints and out-of-plane mechanism of overturning of wedge walls.

PARAMETER 9: Roof typology

The parameter evaluates the thrust action of the roof on the perimeter walls, the effectiveness connection among the roof structure and the walls, the weight of the roof, in terms of seismic mass, and the difference in stiffness and strength compared to the masonry under the roof. In this sense, the scores are assigned in relation to the presence or absence of concrete rings or metal chains, to the permanent load of the roof and to the pitch slope (Figure 2.20). Class A is for no thrust roofs, with or without metal ties or concrete rings. Class B is related to semi-thrusting roofs regardless of the presence of connecting elements, while class C considers thrusting roofs or semi-thrusting roofs, with/without metal ties or concrete rings. In case of thrust exerting roofing without connection systems, class D has to be assigned.

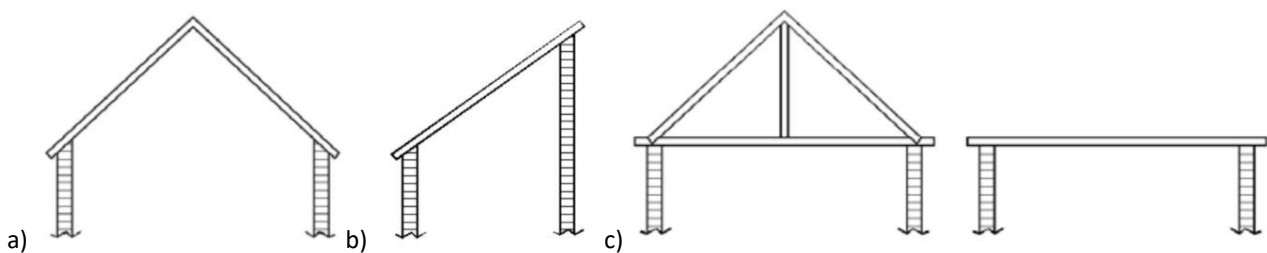


Figure 2.20: Different thrust exerting roofing structural system (a), semi-thrusting (b) and no thrusting roof types.

PARAMETER 10: Non-structural elements

Parameter 10 takes into account the presence of other non-structural elements, like chimneys, additions, ceilings, etc. whose collapse can lead to damage, blockage or injured people. In this sense, the classes are divided in relation to their presence, their dimension and their connection to the vertical structures. Class A considers buildings without fixtures, additions or false ceilings, while class B identifies small chimneys, balconies and attached elements well connected. In case of ceilings and small extensions badly connected, class C has to be assigned. Finally, Class D is related to the presence of chimneys, additions, ceilings, etc. that are poorly connected.

PARAMETER 11: Physical conditions

The last parameter evaluates the overall conservation state of the building, looking at aspects like the crack pattern of the masonry walls (dimensions and width of the cracks), the out of plane of vertical structures and the decay of the material. If the masonry is in good condition with no visible damage, class A is selected, while in case of buildings with scarcely spread capillary lesions, class B is assigned. Class C identifies SUs with modest cracks (width of the lesion: 2-3 mm) and characterized by a state of conservation of the walls, that can determine a significant decrease of resistance. Class D is for buildings that show an out of plane behaviour of the walls, with a significant crack pattern and deterioration of materials.

Table 2. 2: Parameters used in the GDNT (1994) method

Vulnerability Parameters		Short description	Vulnerability Class				Weight
			A	B	C	D	
P1	Organization of vertical structures	Age of the construction and connection typology between the walls	0	5	20	45	1
P2	Nature of vertical structures	Vertical element typology	0	5	25	45	0.25
P3	Qualitative resistance	Walls' shear strength assuming box behaviour	0	5	25	45	1.5
P4	Location of building and type of foundation	Topographical conditions of the ground and foundations characteristics and depth	0	5	25	45	0.75
P5	Floor typology	Quality of floor type considering stiffness and connection with the walls	0	5	15	45	1
P6	Plan regularity	Length/width ratio of the building plan	0	5	25	45	0.5
P7	Height regularity	Mass variation in elevation and the presence of arcades or towers	0	5	25	45	1
P8	Distribution of plan resisting elements	Spacing between walls	0	5	25	45	0.25
P9	Roof typology	Weight and characteristics (thrust) of the roof	0	15	25	45	1
P10	Non-structural elements	Presence, typology and connection to the building	0	0	25	45	0.25
P11	Physical conditions	Masonry quality and cracking scenario	0	5	25	45	1

2.3.2. The Formisano method

With the aim of improving the (GDNT 1994) procedure by considering the SA effect and the interaction among SUs, Formisano et al. (2009) introduced five additional parameters: the position of the SU in the aggregate, the percentage of openings in the façade walls, the presence of staggered slabs, the structural difference between adjoining SUs and the interaction of neighbouring SUs with different heights. Compared to the GDNT method, this approach disregarded, at least in an explicit manner, the qualitative resistance of the walls. Some of the weights (GDNT, 1994) were also redefined. Negative values are assigned to the vulnerability classes with the aim of reducing the vulnerability. The maximum value of the achievable non-normalized I_v is equal to 515.25. For the development of this method, the vulnerability index is compared with a mechanical index ($I_M = D_{max} / D_u$) obtained through the finite element model 3Muri (STADATA 2009). In this sense, non-linear static analyses are performed, considering the ratio between the maximum horizontal displacement (D_{max}) and the ultimate displacement (D_u), of an isolated SU and then within a SA. The model adopted represents the structural features of the construction typology in the urban centre of Sessa Aurunca, South Italy.

PARAMETER 11: Misalignment of openings among the SUs

The misalignment of openings among the SUs can influence the distribution of horizontal actions between the façades, because of the different geometrical dimension of piers and spandrel. The openings percentages (%) among two adjacent structural units defined the vulnerability class: the % of difference for adjacent buildings is less than 5%, the class is “A”, “B” when it is between 5% and 10%, “C” if it is between 10% and 20% and “D” when it is major than 20%.

PARAMETER 12: Masonry disconnections

The parameter evaluates the structural heterogeneity between adjacent buildings, considering that two adjacent SUs can be made with different construction technologies. For class A the building analysed has an equivalent structural system compared the adjacent ones and for class B the SU borders another one built with better masonry characteristics (e.g. a tuff building or a regular block masonry building). On the contrary, for class C the buildings have both typological and structural continuity, while for class D the structure close to the analysed one has worst masonry characteristics (e.g. a tuff building or a chaotic brick masonry building).

PARAMETER 13: Presence of adjacent buildings with different height

The interaction in height of a building with adjacent ones influence in different way the seismic response of the structural unit (Figure 2.21). The optimal condition is given by adjacent building with the same height, where the response is triggered by mutual confinement actions involved (class A). The vulnerability increases, if a building is located adjacent to taller SUs or among a taller buildings and a construction with equal height (class B). The opposite case to class B corresponds to class C and finally if the building is adjacent to two lower buildings it is necessary to assign a class D.

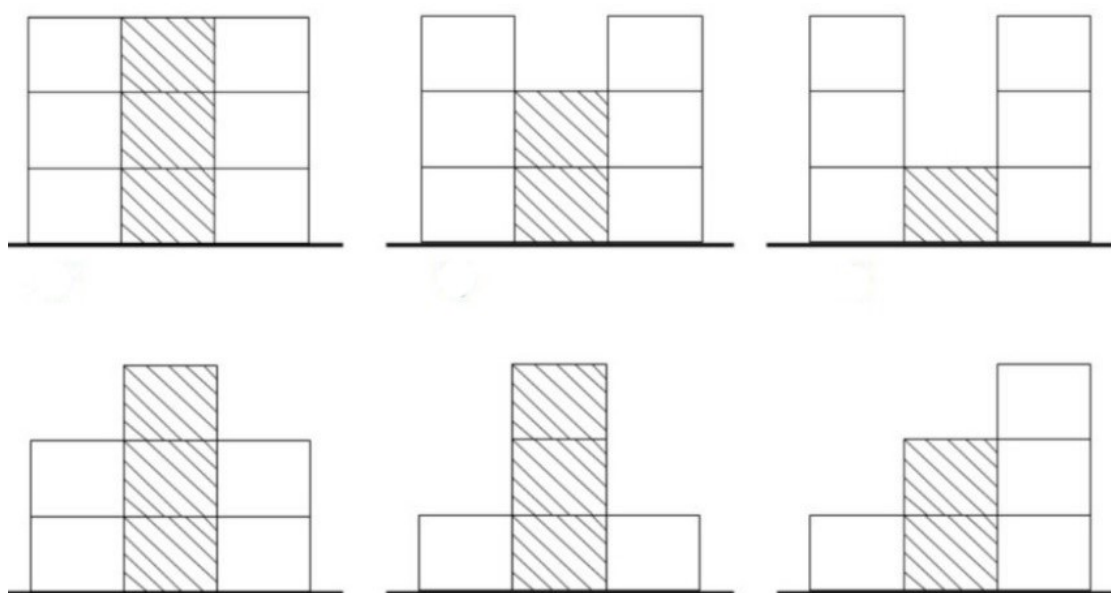


Figure 2.21: Possible altimetric configurations of a structural unit within an aggregate.

PARAMETER 14: Position of the SU in the SA

The planimetric interaction with the adjacent buildings is evaluated, considering the positions of the building within the aggregate: at the top or in the end of an aggregate, inside of an aggregate and or at the corner with two free sides (Figure 2.22). Class A is chosen if the building is located between other buildings and is bounded on three sides. In this case, the adjacent buildings confine the analysed SU, limiting possible displacements and deformations. For class B, the building is bounded on two sides, such as in the case of a building with a courtyard behind, and the confinement effect diminished. In the case of a building on the corner of an aggregate, class C must be assigned. Finally, class D identifies a building at the end of an aggregate.

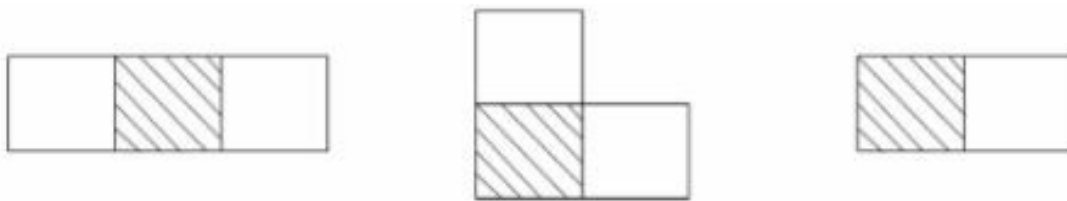


Figure 2.22: Possible positions of a structural unit within an aggregate

PARAMETER 15: Presence and n° of staggered floors

This parameter considers the presence of staggered floors between adjacent buildings, which can generate, during an earthquake, thrusts that aren't balanced by the walls or the hammering effects. The vulnerability associated increases in relation of the number of staggered floors within the aggregate. In case of class A no staggered floors are identified, while for one, two or more than two pairs of staggered floors, the class B, C and D is considered. According to Formisano et al. (2009), it is possible to take into account the presence of staggered floors if there is at least 50 cm of height of difference between them.

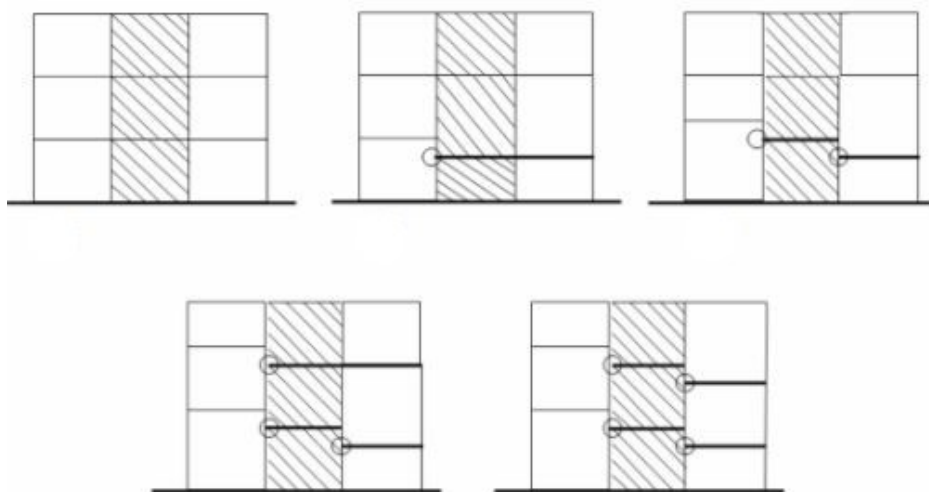


Figure 2.23: Possible positions of staggered floors

All the parameters with the appropriate scores and weights are summarized in Table 2.3.

Table 2.3: Parameters used in the Formisano et al. (2009) method with the additional parameters highlighted in grey

Vulnerability Parameters			Vulnerability Class				Weight
			A	B	C	D	
P1	Organization of vertical structures	Age of the construction and connection typology between the walls	0	5	20	45	1
P2	Nature of vertical structures	Vertical element typology	0	5	25	45	0.25
P3	Location of building and type of foundation	Topographical conditions of the ground and foundations characteristics and depth	0	5	25	45	0.75
P4	Distribution of plan resisting elements	Spacing between walls	0	5	25	45	1.5
P5	Plan regularity	Length/width ratio of the building plan	0	5	25	45	0.5
P6	Height regularity	Mass variation in elevation and the presence of arcades or towers	0	5	25	45	1
P7	Floor typology	Quality of floor type considering stiffness and connection with the walls	0	5	15	45	0.75
P8	Roof typology	Weight and characteristics (thrust) of the roof	0	15	25	45	0.75
P9	Non-structural elements	Presence, typology and connection to the building	0	0	25	45	0.25
P10	Physical conditions	Masonry quality and cracking scenario	0	5	25	45	1
P11	Misalignment of openings among the SUs	% difference of openings in adjacent facades	-20	0	25	45	1
P12	Masonry disconnections	Effect of structural or typological heterogeneity in adjacent SUs	-15	-10	0	45	1.2
P13	Presence of adjacent buildings with different height	Location of the SU in the SA and height's variation	-20	0	15	45	1
P14	Position of the SU in the SA	Number of sides next to other SUs	-45	-25	-15	0	1.5
P15	Presence and n° of staggered floors	Number of staggered floors in the SA	0	15	25	45	0.5

2.3.3. The Vicente method

The purpose of the Vicente method (2010) is to adapt the Italian GDNT form for its application to the Portuguese masonry buildings. In this sense, more detailed analyses are considered within the method, basing on the building stock information, and some criteria are redefined. Two more parameters are introduced to consider other effects in the seismic vulnerability assessment of a structural unit in aggregate. The survey sheet is reorganized into 4 macro-classes of parameters related to the structural building system (P1 to P4), to the irregularities and interaction (P7 to P10), to floor slabs and roof (P11 to P12), to conservation status and other elements (P13 to P14); 3 new parameters (P4, P6 and P7) were also introduced. The maximum value of the achievable non-normalized I_v is equal 650. Like for the Formisano method, also in this case the vulnerability index obtained by the proposed methodology is compared with the results achieved through a numerical model (Robot Office 2004), in which a non-linear analysis evaluates the vulnerability of isolated buildings and aggregates of the historical centre of Coimbra, Portugal.

PARAMETER 5: Number of floors

The criteria of the parameter suggest that taller masonry buildings tend to be more vulnerable and susceptible to second-order effects, when compared to low rise structures. Class A refers to one-floor building, class B to two or three-floors ones, class C to four-five-storeys and class D for more than 5 storeys.

PARAMETER 10: Wall façade openings and alignments

This parameter evaluates the negative effect that the misalignment of openings has in the seismic capacity of the buildings in aggregate. The openings dimensions in the walls and their misalignment, both vertically and horizontally, influence, indeed, the collapse mechanisms either in the plane of the wall or out of the plane. Class A identifies openings with regular size and aligned in height. Class B shows regular or irregular openings, that are horizontally misaligned more than $\frac{1}{2}$ of their height, while for class C the openings, regular or irregular, are vertically misaligned by more than $\frac{1}{2}$ of their width. Finally, for class D the openings are large, regular or irregular and totally misaligned horizontally or vertically

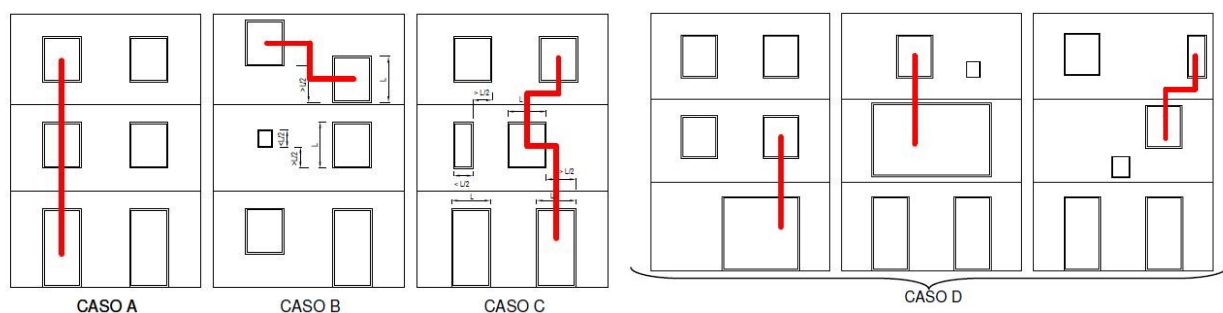


Figure 2.24: Schemes for identify the possible Classes of parameter 15 and according to the possible configurations of the openings misalignments and dimensions.

Table 2.4 shows the survey sheets used in the Vicente form.

Table 2.4: Parameters used in the Vicente (2010) method

Vulnerability Parameters		Short description	Vulnerability Class				Weight
			A	B	C	D	
1. Structural building system							
P1	Type of resisting system	Construction age, quality of walls' connection	0	5	20	50	0.75
P2	Quality of the resisting system	Vertical Element Typology	0	5	20	50	1
P3	Conventional strength	Walls' shear strength assuming box behaviour	0	5	20	50	1.5
P4	Maximum distance between walls	Spacing between walls	0	5	20	50	0.5
P5	Number of floors	Number of floors in the building	0	5	20	50	1.5
P6	Location of building and type of foundation	Topographical conditions of the ground and foundations characteristics and depth	0	5	20	50	0.75
2. Irregularities and interaction							
P7	Aggregate position and interaction	Position of the SU in the SA	0	5	20	50	1.5
P8	Plan configuration	Length/ width ratio of the building plan	0	5	20	50	0.75
P9	Height regularity	Mass variation in elevation and the presence of arcades or towers	0	5	20	50	0.75
P10	Wall façade openings and alignments	Influence of the misalignment	0	5	20	50	0.5
3. Floor slabs and roofs							
P11	Horizontal diaphragms	Quality of floor type considering stiffness and connection with the walls	0	5	20	50	1
P12	Roof typology	Weight and the roof typology (thrust)	0	5	20	50	1
4. Conservation status and other elements							
P13	Fragilities and conservation state	Masonry quality and cracking scenario	0	5	20	50	1
P14	Non-structural elements	Type and characteristics of the non-structural elements	0	5	20	50	0.5

2.3.4. The Ferreira method

This approach is easy and quick to apply, because it is composed by only five parameters, almost qualitative, and allows assigning a vulnerability index to the entire aggregate. The simplified approach of this method allows its applicability as a preliminary screening of the seismic vulnerability of aggregates. It was tested for the first time in an existing aggregate, composed of 13 limestone masonry buildings, located in the historic city centre of Coimbra, Portugal. Later, it is also applied to the Italian building stock of Castelnuovo, Italy, after the seismic event of the 2012 (Boschi 2015), to verify its reliability for Italian urban centres.

PARAMETER P1: Quality of the masonry fabric

This parameter assesses the masonry quality of the SUs of the analysed aggregate, with the aid of sub-classes of masonry. Vulnerability classes are selected in relation to the type and heterogeneity of the masonry present

in the structural system. If the quality and the homogeneity of masonry decreases, the values of the score increase. Figure 2.25 shows the rules for the vulnerability class assignment of parameter 1, as well as the sub – classes of masonry typologies.

A	More than 75% of buildings belong to subclass Sc1.
B	Less than 25% of buildings belong to subclass Sc3 and Sc4 and more than 25% of buildings belong to subclass Sc2, Sc3 and Sc4.
C	Less than 25% of buildings belong to subclass Sc4 and more than 25% of buildings belong to subclass Sc3 and Sc4.
D	More than 25% of buildings belong to subclass Sc4.

Parameter P1: Subclass definition

Sc1	Brick masonry of good quality and fabric (solid bricks or hollow bricks with less than 45% of voids). Well-tailored stone masonry with homogeneous size units. Well mortared irregular stone masonry, well interlocked, presenting transversal elements in the connection between leafs.
Sc2	Brick masonry (less than 45% of voids). Well-tailored stone masonry with little homogeneous units. Irregular stone masonry with transversal connection between leafs.
Sc3	Brick masonry of poor quality with connection and bricklaying irregularities. Stone masonry with no tailored and heterogeneous units. Irregular stone masonry well mortared and well interlocked but with no transversal connection elements.
Sc4	Poor quality brick masonry incorporating stone and brick fragments. Loose stone masonry. Rubble stone masonry, with no transversal connection and poorly mortared.

Figure 2.25: Vulnerability class and sub-classes of the masonry type definition for Parameter P1 according to the Ferreira Form.

PARAMETER P2: Misalignment of openings

This parameter evaluates the presence of horizontal misalignments of openings and staggered floors. The assignment of the vulnerability classes depends on the ratio between the number of staggered floors and the total number of the adjacent floors, according to the criteria defined in Figure 2.26.

A	Less than 25% of cases of horizontal misalignment of openings between adjacent buildings.	(or)	Less than 25% of cases of floor misalignment between adjacent buildings wherein one of the buildings presents concrete floors and/or structure.
B	More than 25% and less than 50% of cases of horizontal misalignment of openings between adjacent buildings.	(or)	More than 25% and less than 40% of cases of floor misalignment between adjacent buildings wherein one of the buildings presents concrete floors and/or structure.
C	More than 50% and less than 75% of cases of horizontal misalignment of openings between adjacent buildings.	(or)	More than 40% and less than 60% of cases of floor misalignment between adjacent buildings wherein one of the buildings presents concrete floors and/or structure.
D	More than 75% of cases of horizontal misalignment of openings between adjacent buildings.	(or)	More than 60% of cases of floor misalignment between adjacent buildings wherein one of the buildings presents concrete floors and/or structure.

Figure 2.26: Vulnerability class definition for Parameter P2 according to the Ferreira Form.

PARAMETER P3: Irregularities in height

Parameter 3 takes into account the irregularities of the building heights with respect to an average height of the aggregate. The criterion adopted for the class definition deals with the evaluation of the ratio between the sum of the number of floors missing enter two adjacent structural units and the total number of floors of a SA (Figure 2.27). For class A this ratio must be lower than 0.2, for class B lower than 0.5 and for class C lower than 0.8. On the contrary class D identify buildings with a ratio equal or higher than 0.8.

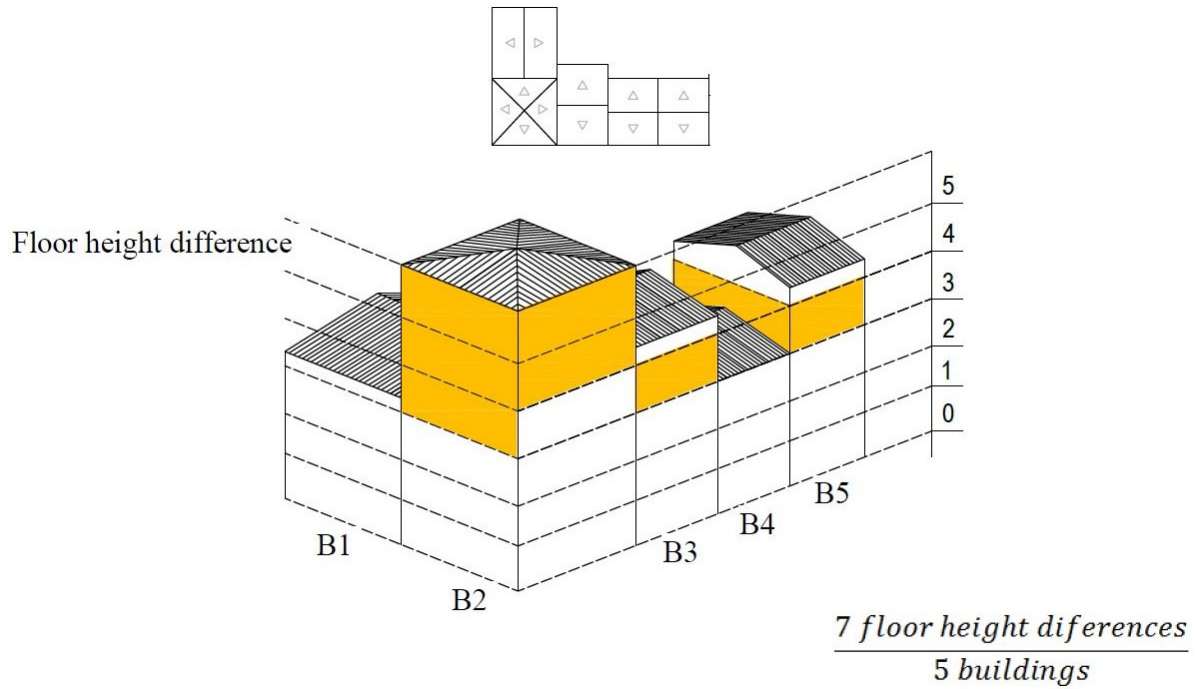


Figure 2.27: Scheme for the evaluation of the ratio for the definition for Parameter P3 according to the Ferreira Form.

PARAMETER P4: Plan geometry

This parameter evaluates the plan irregularities of the structural aggregate, using the relationship between the area, A , and the perimeter, P , of the plan shape of the aggregate (Figure 2.28). As the ratio of the area to perimeter of the aggregate decreases the irregularity is evident and therefore the vulnerability increases, according to the following formulation:

$$P_4 = \frac{16A}{P^2} \quad [2.11]$$

For class A, this relationship has to be lower than 1, while for class B is has to be in the range of 0.75 and 1. If P_4 is among 0.5 and 0.75, class C has to be assigned and for values less than 0.5 class D is identified.

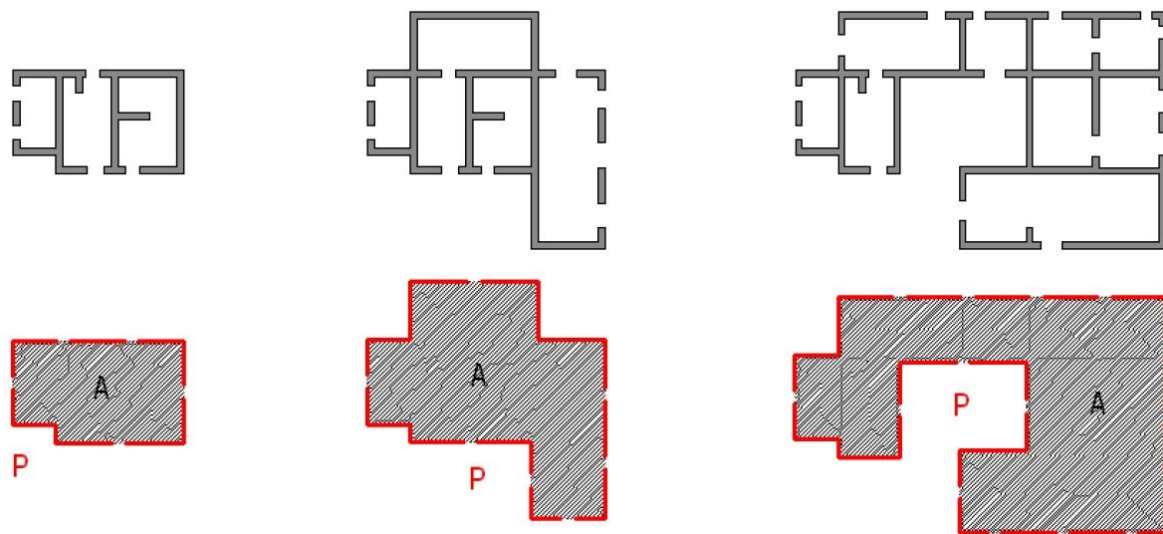


Figure 2.28: Graphical scheme for the evaluation of the aggregate geometry in plan according to the Ferreira Form.

PARAMETER P5: Location and soil quality

This parameter assesses the quality of the ground foundation and the slope of the aggregate: as the quality of the soil decreases and as the slope increases, it is necessary to assign worst classes for vulnerability evaluation. Figure 2.29 shows the rules to follow for the definition of the vulnerability class in relation to parameter 5.

A	Aggregate founded on rock or on a coherent soil with less than 10% slope. Located in areas with no special constraints and no gaps.
B	Aggregate founded on rock or on a coherent soil with a slope between 10 and 30%. Landfill soils with gaps.
C	Aggregate founded on rock or on a coherent soil with a slope between 30 and 50%. Landfill soils with probable impulses.
D	Aggregate founded on rock or on a coherent soil with more than 50% slope or on heterogeneous soil with more than 50% slope. Located in ravine or cliff. Possible soil liquefaction, soil slip (landfill and alluvial layers), layered soil heterogeneity, soft and loose clay soil, landfills

Figure 2.29: Vulnerability class definition for Parameter P5 according to the Ferreira Form.

Table 2.5 summarizes the parameters necessary for the vulnerability index evaluation according to the Ferreira form.

Table 2.5: Parameters used in the Ferreira et al. (2012) method

Vulnerability Parameter		Short description	Class Score				Weight
			A	B	C	D	
P1	Quality of the masonry fabric	Type of masonry among the SUs in the aggregate	0	5	20	50	1.50
P2	Misalignment of openings	Ratio between the number of staggered floors and the total number of the adjacent floors.	0	5	20	50	0.50
P3	Irregularities in height	Difference in height among adjacent structural units	0	5	20	50	0.75
P4	Plan geometry	Plan irregularity, using a relationship between the area and the perimeter	0	5	20	50	0.75
P5	Location and soil quality	Quality of the ground foundation and the slope of the aggregate.	0	5	20	50	0.75

2.4. Conclusions

The study of historical centres is a complex, multidisciplinary procedure that requires a transversal knowledge of many fields, such as the dynamic behaviour of masonry structures, the survey methods and material properties, the historical development of the construction typology analysed, a deep knowledge of construction and retrofit techniques, performed in the current day and in the past. The literature review shows the absence of a consistent and codified tool for the analysis of masonry aggregates in old urban centres, where most of the information obtained, derives from previous experiences performed on several case studies since the 1990s.

On the other hand, there are several methodologies, developed in the last five decades, for the assessment of seismic vulnerability, even if they are not strictly linked to the study of historical centres. The choice of a method depends on the results anticipated in relation to the resources available. Empirical/Expert Base Methods are less accurate, but with a small amount of data, are able provide a first indication of the seismic behaviour of the analysed structure. On the contrary Analytical/Mechanical Methods are more reliable, but the level of details required (structural, materials, soil, etc.) does not allow their large-scale application. Hybrid Methods try to overcome the limits of the previous methods. VULNUS and FaMIVE procedures represents and evolution of the observational approaches, but the nature of the results obtained is not clear and affected by the same uncertainties of Empirical/Expert Base methods, such as the quality of the available information.

The geographic applicability, the evaluation of the surrounding environments following an earthquake (roads, escape routes, etc.) and their easy application, make Empirical/Expert Base methods suitable for the assessment of seismic vulnerability of historical centres on a large scale. According to the research's objectives four methods are analysed in detail: the GDNT, the Formisano, the Vicente and the Ferreira form. The GDNT represents the base on which the other vulnerability index methods are developed. Since its use is limited to analysis in isolation, the Vicente and Formisano forms add new parameters that take into account the aggregate effect in the dynamic behaviour of masonry SUs in aggregate. The Ferreira form, differs from the previous methods, provides a preliminary screening of the seismic vulnerability of masonry aggregates.

The calibration of these methods on specific construction typologies has a direct effect on the value attributed to the weights of different parameters. This aspect must certainly be taken into account in the reliability evaluation of the seismic damage scenarios, developed through these methods.

The knowledge acquired in this phase is considered as a base for the development of the research activity and provides several insights and reflection, that are encountered and addressed in the following chapters.

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CHAPTER 3. The case study of Campi Alto di Norcia

Campi Alto di Norcia is an old city centre in Italy, Umbria Region, hit by the Central Italian Earthquake in 2016 and assumed as a case study for this research activity. The main characteristics of the buildings and of the urban organization are described in detail to have a precise database to develop different type of analyses. The information obtained through different research agreements, signed for this work activity, give the possibility to evaluate different aspects and provide a clear knowledge of the seismic behaviour of masonry aggregates.

Chapter outline

3.1 Introduction (methodology)

3.2 Historical background

3.3 Analysis of the built heritage

3.3.1 Structural aggregate: General features and urban organization

3.3.2 Structural Units: Main features and classification

3.3.3 Materials and masonry typologies

3.3.4 Retrofitting systems

3.4 Conclusions

3.5 References

3.1. Introduction (methodology)

The selected case study is the medieval town of Campi Alto di Norcia in Valnerina (Umbria), Italy, reliable for the wide structural typologies of Central Italy, and stroke by the earthquakes of August 24th and October 30th, 2016.

The choice of this village is due to several reasons: the principal one is that at the time of the inspections carried out - autumn 2018 and summer 2019 - the medieval town of Campi Alto di Norcia was still considered by the Italian Institutions (Umbria Region, Civil Protection and Government) a red zone, i.e. an area subject to a high risk and banned to the public. Buildings, streets and urban environment had, therefore, the same level of damage achieved after seismic events of 2016 and no safety measures had been carried out, with the exception of the main church and its bell tower. In this sense, Campi Alto di Norcia represents a robust and consistent sample space; a solid base for seismic research activities (calibration models, damage survey, reconstruction projects, etc.) and for the verification of the seismic behaviour of masonry aggregates, theoretically analysed in the first work phase.

In this sense, the contribution and experience gained in this historical centre by the Department of Architecture of the University of Ferrara, with which a research agreement was signed for this work, was fundamental for the acquisition of different information (Baruffaldi and Marchi 2018), (Arzilli and De Venz 2018).

Another reason linked to the choice of this village as a case study is, that it was subjected to different seismic events throughout its history (analysed in detail in chapter 4). Among the latter, the damage caused by earthquakes of 1979 ($I_{MCS} = 8$) and 1997 ($I_{MCS} = 6$) was deeply studied and several projects for reconstruction (P.D.R. 1982), (R.U. 9 2003) and scientific researches were carried out (AA.VV. 1980), (Ballardini 1998), (Benetti 1998), (Cardani 2003). This gave the opportunity to have a clear knowledge of the conditions of the premises before the earthquakes and to compare the current damage scenario. An agreement undersigned for this research activity between the University of Florence and the Umbrian Region provide several information on consolidation and reconstruction projects of the buildings in Campi Alto di Norcia. In this sense, it was possible to evaluate the reliability and the performance of the retrofits, following the last seismic events.

The study of the historic centre of Campi Alto di Norcia was organized on several levels: starting from the urban setting, the detail of survey is increased to the building scale.

The analyses of the city organization take into account the streets arrangement and the position of the constructions, according to the land morphology and the historical development. An analyses was performed through the research of the historical sources, such as the Gregorian Cadaster and archival documents (AA.VV. 1977), (Chiuni 1986), (AA.VV. 1993). The data were post processed and verified during the inspections to define the historical layering of the old city centre.

The survey of the masonry buildings and aggregates was performed with specific survey forms, developed on the base of the survey sheets of the Department of Civil and Environmental Engineering of the University of Florence (Boschi 2015). These survey sheets summarize and organize in a standard way the information about the buildings gained during the inspections. In the first part, the position of the building in the historical centre is evaluated through aerial photos and plans and an identification number is assign. In the second part, the principal structural features are defined: masonry characteristics, roof and floor typologies, presence of retrofits, structural details (i.e. vaults, stairs, etc.). A series of drawings of the plans and façades of the building are shown with their geometric values. In the last part, an estimation of the possible weights of the floors is performed, based on the information collected in the previous sections. An example of the survey sheets, adopted in this phase, is provided as an important output of the thesis in Annex A.

This phase of the research activity allows also to evaluate guidelines for the survey and analysis of historic centres. Figure 3.1 shows the methodology applied: following standard procedure, all the activities are defined in sequence and the main aspects to be analysed are highlighted.

The buildings survey is also focused on providing the best possible quality of the data, used in the Empirical/Expert base Methods, described in the previous chapter, for the definition of the seismic vulnerability index. To improve visualization and analysis, the relevant features of the buildings, are integrated into a GIS tool.

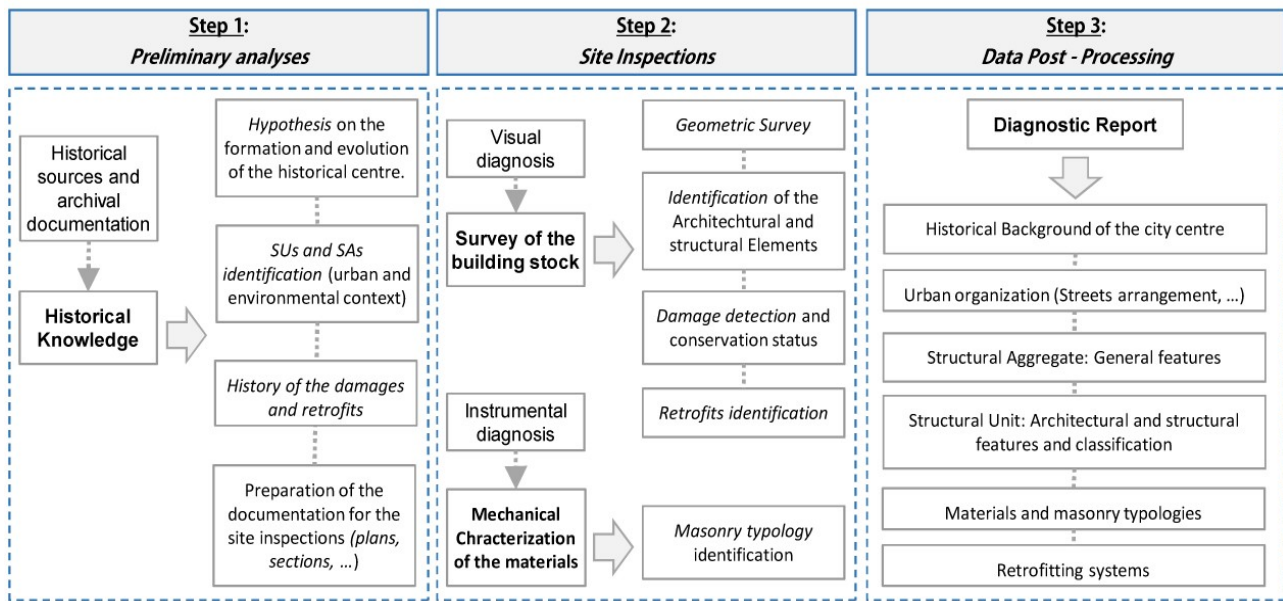


Figure 3.1: Methodology applied for the identification of the historical centre of Campi Alto di Norcia

3.2. Historical background

Around the 6th century BC, it is possible to have historical evidence, consisting of the remains of a necropolis and a temple, of several settlements in the Castoriana and Campiana valleys, where Campi Alto di Norcia is located. At that time, before the Roman conquest, this territory was the border between the Etruscans and Umbrians and represented the natural crossroads between Camerino and Ascoli. (Manconi and De ANGELIS 1987). The economy was essentially agricultural and the historical sources are few and don't provide relevant data.

Further information start from the sixth century AC, *Dialogues of Saint Gregory* (Calati and Stendardi 2001), as the valleys become a place of prayer and where it is possible to practice a hermit's life. The then bishop of the city of Spoleto (Umbria), Saint Spes, established in this period several hermitages and monasteries throughout the valley. Saint Eutizio, following the teachings of the bishop, began the construction of an important abbey in the valley (Figure 3.2), still visible nowadays and which takes its name. Between the 10th and 12th centuries, the feud of the abbey grew, increasing the population of the villages in the valley: several areas felt under its control and influence, including the first settlement of Campi Basso¹, located at that time at the base of the hill (Pirri 1960), (Destro and Giorgi 2001).

¹ The name of Campi Basso or Campi Vecchio is introduced to specify the location of the first settlement and to distinguish it from Campi Alto di Norcia, the case study of the research activity. According to historical documents, the position of Campi basso was in the area of the church of San Salvatore. Nowadays the latter is completely destroyed after the 2016 earthquake and only some perimeter walls and the grounds of the structure are visible.



Figure 3.2: Saint Eutizio Abbey before the 2016 earthquake (a) and location of Campi in 1600, highlighted in red, according to a historical map of the Dukedom of Spoleto (Bianchi and Simona 2017).

The abbey had a significant importance until the 12th century, when it began to lose political power due to the centralizing trends of the church and the expansionist aims of the nearby towns, but remained an important religious and cultural centre. In 1227, the city of Norcia took possession of the Presenzano castle, starting the conquest of the territories and properties of the abbey, including Campi Basso (Patrizi Forti 1869). Around 1280 and 1890, the government of Norcia decided to ground a new settlement of Campi Basso on the slope of the nearby hill (Macchialunga Mount) and to transform it into a fortified urban centre: The castle of Campi Alto. Wealth and prosperity grew in that period, testified by the number of churches and buildings that were built within the perimeter walls of the village. As the importance of the castle of Campi Alto grew, the area around Campi Basso lost importance and became more and more depopulated (Figure 3.3).



Figure 3.3: View of the church of San Salvatore with Campi Alto di Norcia on the background, before the 2016 earthquake (photo made by Silvio Sorcini) (a); Medieval paint of Saint Andrea, that metaphorically embraces the castle of Campi Alto, located in the church of Saint Andrea (Cardani 2003).

The XIV and XV centuries were characterized by wars for the control of the territories of the valley between the town of Norcia, the neighbouring cities and the Pope. The mercenaries of the kingdom of Naples called by Eugenio IV against the militias of Francesco Sforza met in the area (Pirri 1960).

After these events there was a social and economic rebirth in the valley: new buildings and monasteries were built and several churches expanded (Barberini 1610), (Garibaldi 2018),

In 1639 a convent of Benedictine nuns, dedicated to Saint Ursola, was also built inside the city of Campi Alto. In this period, the main testimony regarding Campi Alto is testified by the Bishop of Spoleto, Carlo Giacinto Lascaris, who undertook a pastoral visit of these zones in the years 1712-1713, to check the condition of the churches after the earthquake of 1703 (Lascaris 1712). After this seismic event different part of Campi Alto were rebuild as shown by the maps of the Gregorian cadastre of 1820 (AA.VV. 1820), which provide a precise indication of the position of all the structures present within the walls in that period.

After the earthquake of 1859, Campi Alto started to lose its original function and several residents moved to the base of the hill, along the new main road network, which connected the other villages of the valley. The maps of Campi Alto of 1939 (AA.VV. 1939) shows that many buildings were abandoned and not rebuild. In that period the convent of the nuns dedicated to Saint Ursula with the adjoining Church of San Giovanni went into ruins, which are still visible today, among the dense vegetation. The walls of the castle and the three main gates were lost, with the exception of a small portion in the south and east part of the historical centre.

Before the seismic events of 2016, in Campi Alto di Norcia there was three churches of the six existing within the walls of the town and the built heritage was formed by 72 residential buildings, besides other structural units used as storage areas and cellars (Figure 3.4).

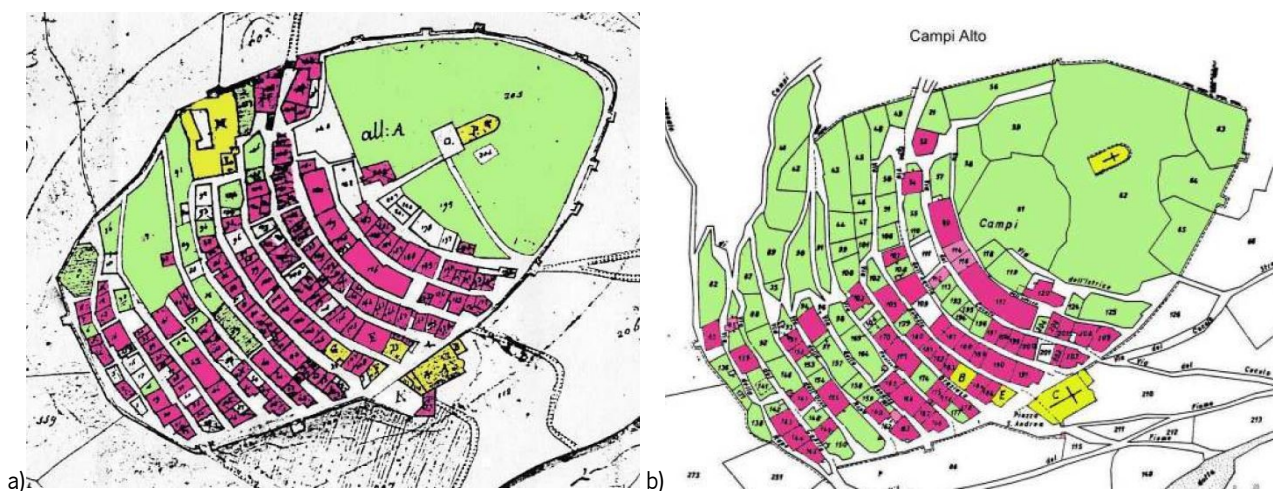


Figure 3.4: The map of Campi Alto di Norcia, according to the Gregorian Cadastre of 1820 (a) and 1939 (b). In yellow the churches are highlighted, while the pink colour indicates the structural units of the village (Cardani 2003).

3.3. Analysis of the built heritage

The urban organization and development of a human settlement is determined by the morphology of the territory, the main economic activity and its strategic position. These characteristics involve the application of different solutions in terms of shape of the structures and materials used for their construction. In the historical towns, while churches or monumental buildings may present special architectural solutions, most of the common masonry structures are built with materials found in nearby quarries.

Campi Alto di Norcia, even with the gaps caused by more ancient collapses, is remained intact in its fundamental lines: a medieval castle with a triangular shape, developed perpendicular to the slope of the hill, partially surrounded by a perimeter walls. The entrance gates are located laterally, in a halfway position.

Each structural unit is identified by a number, taken from the Gregorian Cadastre, to distinguish it from the others.

3.3.1. Structural aggregate: General features and urban organization

The building heritage of Campi Alto di Norcia covers an area of approximately 3,5 ha with 32 masonry aggregates (SAs) resulting in a total of 75 structural units (SUs) including three churches (i.e. Madonna della Piazza, Sant'Andrea and Santa Maria delle Grazie), fully destroyed by the 2016 earthquake sequence. According to the site morphology, SAs develop on three different levels, perpendicular to the slope of the hill on which the settlement is located. The first level has the entrance in correspondence of the downstream road and the top floor at the level of the upstream road. The different levels and SAs are then connected through an internal organized system of staircases (Figure 3.5).



Figure 3.5: View of Campi Alto di Norcia after the 2016 earthquakes (a); Identification of Structural Units, highlighted in red and identified with different IDs, within Campi Alto di Norcia (b).

The construction heritage of Campi Alto di Norcia is made up of both isolated masonry buildings and SAs arranged in rows, following the topography of the land. The gross building total volume and area are respectively

about 15000 m³ and 3065 m². The SAs differ in terms of number, shape and height of SUs constituting them. In volume, isolated buildings cover the 29% of the whole stock (resulting in a volume of about 4243 m³), while external (end position) and internal (in between) SUs cover, respectively, the 44% and 27% of whole constructions (i.e. 6577 and 4035 m³) (Figure 3.6). The number and height of the floors, the maintenance conditions and the materials adopted (both for vertical and horizontal structures) are different for each structural unit. Storage areas and cellars were not further considered in the assessment procedure, therefore reducing the number of analysed SUs to (only) 67.

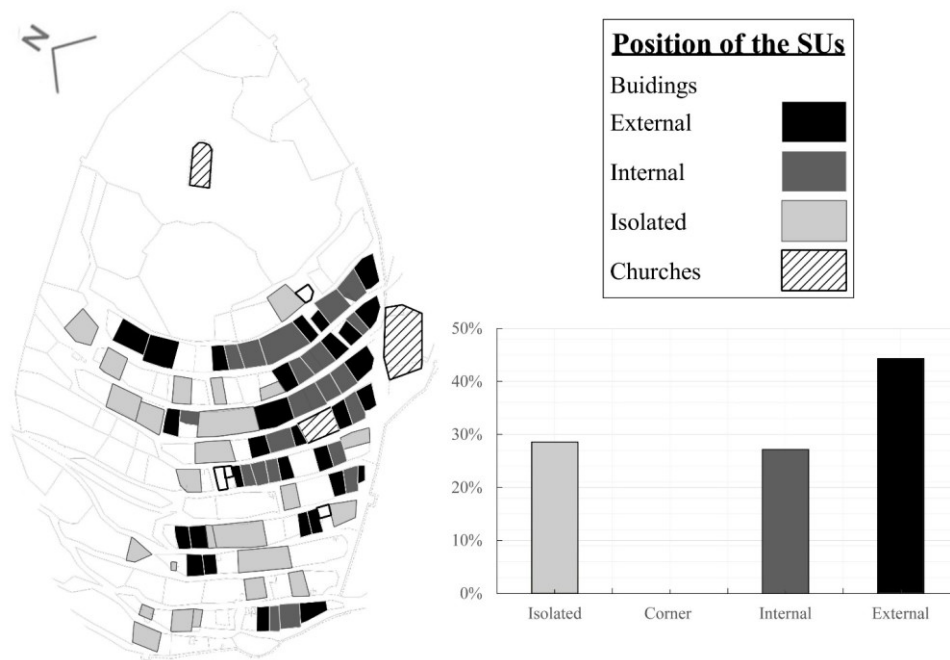


Figure 3.6: Position and typologies of SUs in Campi Alto di Norcia.

3.3.2. Structural Units: Main features and classification

The typical SU is organized on three storeys: the ground floor – partially underground following the hill's slope – and two additional upper floors. A barrel masonry vault carved into the rock is usually present at the ground level, while the storeys of the other floors, as well as the roof, show the typical wooden structure sometimes replaced by reinforced concrete elements in case of retrofit interventions over the years (Figure 3.7). The average inter-storey height of the single SU is about 3.0 m. Masonry walls show different thickness along the height, with average values around 120 cm at the ground floor and reductions up to 70 cm in the upper levels. About the 16% of the buildings surveyed are one-storey constructions, often representing the rest of medieval houses.

Two, three and four-storey buildings represent respectively about the 7%, 66% and 11% of the whole constructions (Figure 3.8). Normally, the organization of the SUs is the same, with a barrel vault at the first level and wooden or concrete slabs in the upper ones.

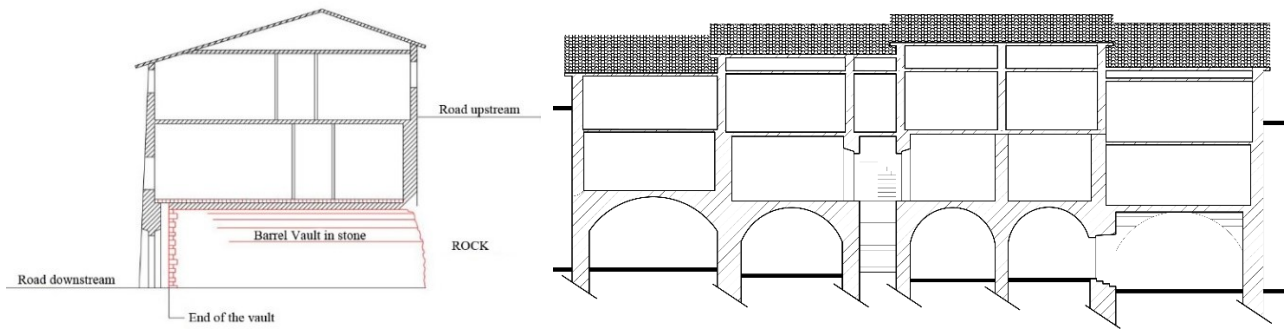


Figure 3.7: Schematic representation of a typical Structural Aggregate (SA25 is shown) in Campi Alto di Norcia (left image adapted from Valluzzi et al. (2009))

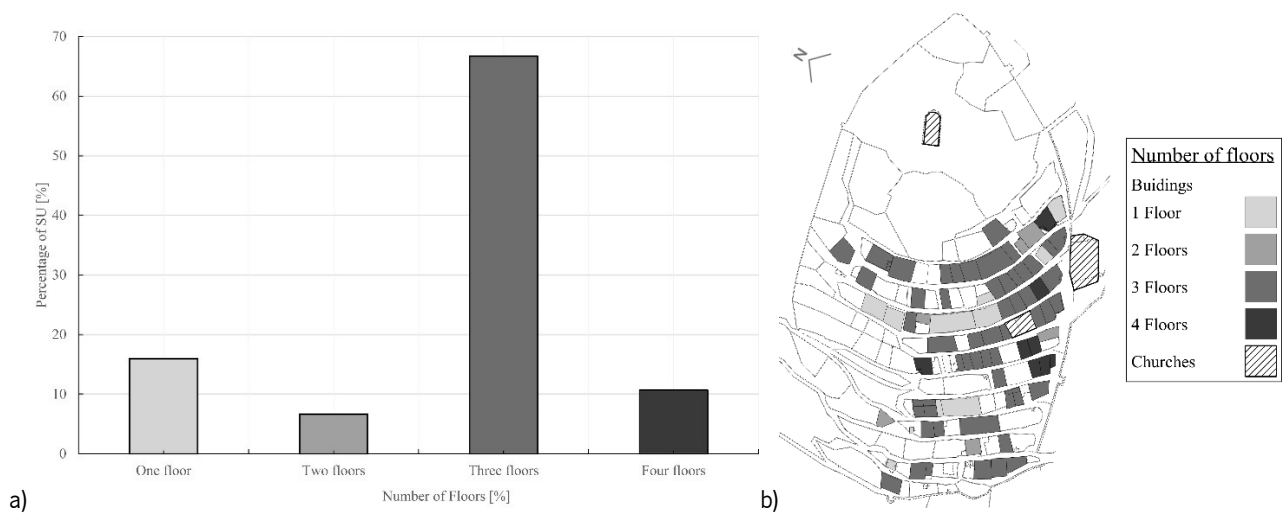


Figure 3.8: Histogram (a) and map (b) of the distribution of the floor number for each SU in Campi Alto di Norcia. Horizontal storeys and roof typologies

Two main categories of horizontal storey typologies can be recognized, i.e. rigid, semi-rigid or deformable ones, in relation to the presence (or the lack) of concrete slabs and crossed/single timber joints. Steel ties to connect timber floors to walls and/or masonry opposite walls are frequently introduced (Figure 3.9b), providing the difference between '*bad*' and '*good*' connection to the vertical bearing system. Less buildings have a deformable slab with a good (5%) or bad (2%) connection to the walls, while semi – rigid floors well connected and rigid bad connected are respectively 5% and 11%. The majority of the SUs detected have semi-rigid floors and are bad joined with the perimeter walls (30%), as well as rigid floors good connected (47%). The first floor in 86% of the buildings is made with a stone barrel vault (Figure 3.9a).

Concerning the roof system, buildings in Campi Alto di Norcia have typically a mono-pitched or gable roof structure, being about the 80% realized with steel/concrete beams and concrete slabs, avoiding horizontal thrusts acting on the external walls; the remaining the 20% shows wooden structures, completed by a roof overhang placed on the perimeter of the aggregate. The roof typologies are therefore divided in '*Heavy*' or '*Light*' if made of concrete/steel or wood, and in '*Thrust*' or '*No Thrust*' when they span perpendicular or parallel to the main façade (Figure 3.10a and Figure 3.10b).

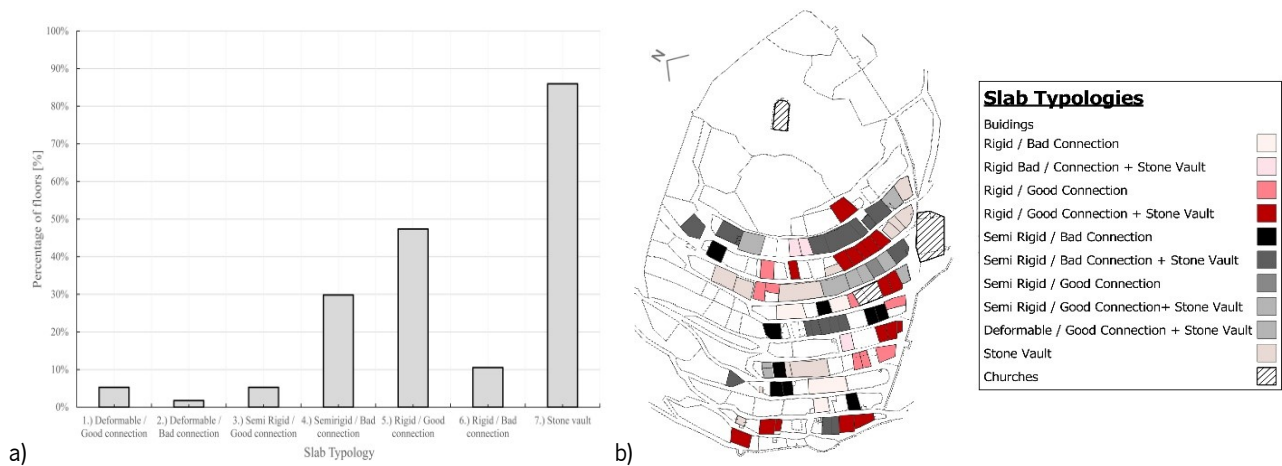


Figure 3.9: Distribution of floor's typology (a) and distribution of the roof typology (b).

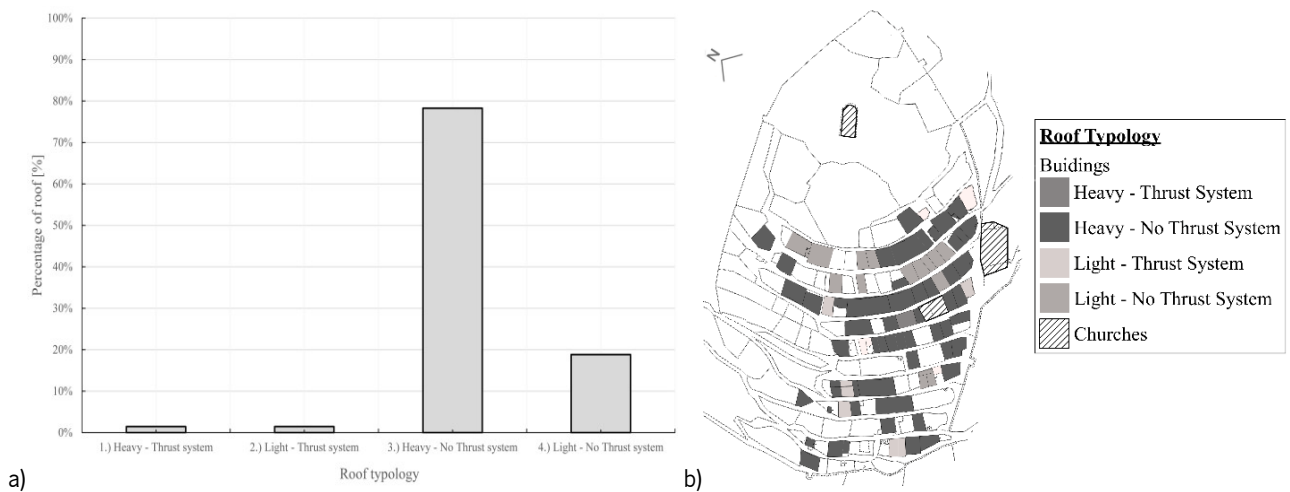


Figure 3.10: Histograms regarding the percentage of the slab (a) and roof (b) typologies

The stratigraphy of the floors and roofs are defined through in situ inspections to evaluate an order of magnitude of their weight per square meter, for the following analysis (Table 3.1).

Table 3.1: Slab and roof typologies identified in the different SUs of Campi Alto di Norcia.

Slab and roof typologies in Campi Alto		weight per square meter
	Barrel vault in stone	
	Description	Value
G1	Self-weight structural	5.45 KN/mq
G2	Carried Permanent Loads	2.65 KN/mq
	Total	8.10 KN/mq

**Concrete Slab**

	Description	Value
G1	Self-weight structural	3.12 KN/mq
G2	Carried Permanent Loads	2.73 KN/mq
	Total	5.85 KN/mq

**Wood Slab**

	Description	Value
G1	Self-weight structural	1.70 KN/mq
G2	Carried Permanent Loads	2.43 KN/mq
	Total	4.13 KN/mq

**Concrete Roof**

	Description	Value
G1	Self-weight structural	3.12 KN/mq
G2	Carried Permanent Loads	0.95 KN/mq
	Total	4.07 KN/mq

**Wood Roof**

	Description	Value
G1	Self-weight structural	1.70 KN/mq
G2	Carried Permanent Loads	0.61 KN/mq
	Total	2.31 KN/mq

The *percentage of openings* of the facades is another parameter that influence the quality of the building system of a SU, in terms of wall resistance. In the same way, the different areas of opening among adjacent façades could affect the distribution of horizontal actions in the SA. The openings typologies are identified in ranges, as the ratio between the value per square meters of the voids and the area of the façades of the SUs (Figure 3.11). In case of total or partial collapse of the building or in case of single storage structures, it was not possible to detect the exact percentage of openings and, therefore, the number of SUs analysed decreased from 67 to 47. The amount of SUs with a percentage of openings less than 5% are 2 and represent the 4% of the total. Buildings with openings between 5% and 15% are the majority (57%), while 18 SUs have medium openings (between 15% and 50%). No buildings have a percentage of openings greater than 50%.

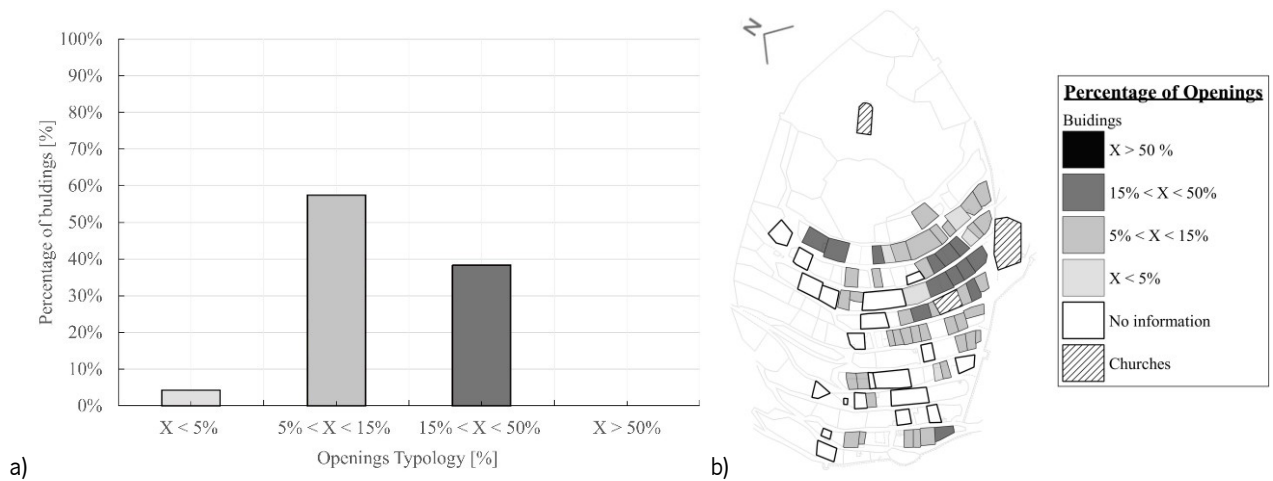
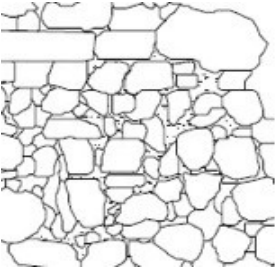
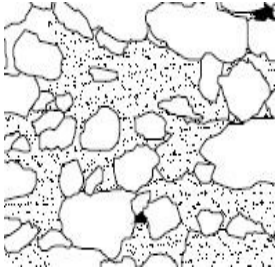


Figure 3.11: Distribution of the different percentage of openings in the façades of the buildings in Campi di Norcia.

3.3.3. Materials and masonry typologies

The classification of the masonry typologies of the SUs is performed looking at their conservation condition (i.e. presence of deterioration phenomena, cracking patterns, etc.) and at the disposition of resistant blocks; of course, if needed, to the specific masonry typology can be then associated a conventional strength (i.e. mechanical properties) basing on indications provided by current standards and/or scientific literature database (Zuccaro et al. 2015), (Borri and De Maria 2019). Four different masonry typologies, all belonging to the general Irregular Masonry typology have been recognized; only in a few cases, e.g. in presence of new SUs or in retrofitted portions, the inner face of the walls is composed by regular brick blocks, while external face consists of irregular masonry. The only case of regular stone masonry is the Church of Saint Andrea, which is not the purpose of the research. The masonry typologies identified are summarized in Table 3. 2 *Table 3.1*.

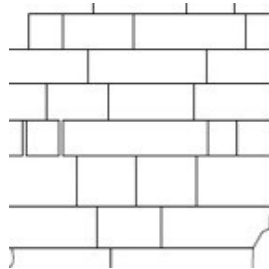
Table 3. 2: Masonry typologies identified in the different SUs of Campi Alto di Norcia.

Masonry Typology		Description
		Irregular Masonry typology with rough blocks Rough-hewn material; this category includes other sub-categories, with differences related to the amount of mortar used for the walls. These masonry typologies have been detected in SUs where the original masonry was merged with a new one.
		Irregular Masonry typology with irregular blocks Masonry walls made of uncut material, i.e., with no evident courses. This type of masonry, represents the majority of masonry in Campi Alto di Norcia, where the wall is composed of two weakly-connected leaves. Typically, the outer face is not plastered and presents an irregularly arranged masonry, whereas the inner one is plastered.



Irregular Masonry type with irregular packed blocks

Mixed structures made of both rough and squared material. This is different from the previous one because of less mortar amount is present in the masonry. Again, this type of masonry, together with the previous one, is widely diffused in Campi Alto di Norcia.

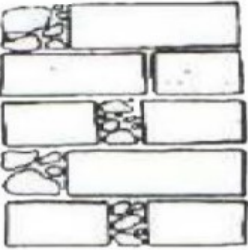


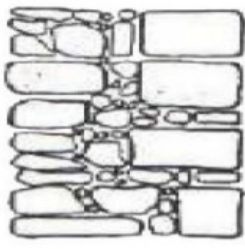
Regular Masonry typology

Squared material with regular courses: established entirely from squared ashlars, laid with a thin lime bed. These masonry types have been detected in new SUs or in portions of retrofitted masonry.

The damage caused by the last seismic event, analysed in detail in chapter 4, allow to evaluate the geometry and the internal disposition of the masonry walls of the buildings. Three types of double – leaf walls, summarized in Table 3.3, are identified in Campi Alto di Norcia according to Giuffrè (1993): Case A, B and C. The difference between them regards the capacity of masonry to ensure a sufficient transverse connection inside the wall, thanks to the internal disposition of the bricks or the presence of wood/stell ties. Bad connected double – leaf walls increase the possibility of instability for each external leaf, which appears as a swelling, and the seismic out-of-plane behaviour (Griffith et al. 2003). In general, the building stock of the case study is characterized by SUs with double – leaf walls, poor connected, and by SUs with retrofitted walls, that reduce or prevent instability phenomena of masonry. Since it was not possible to carry out surveys in all the wall sections of the buildings, the distribution of the three typologies is not clear.

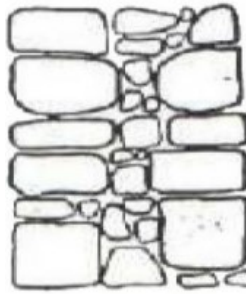
Table 3.3: Section walls Masonry typologies identified in the different SUs of Campi Alto di Norcia.

Section Walls	Description
 	<i>Case A: Regular/Irregular Masonry type, transversely connected</i> Double – leaf wall made by regular/irregular masonry type, where the connection is determined by a good transverse arrangement of masonry and the presence, inside the wall, of medieval timber bindings or modern steel ties. Instability of masonry wall is less probable.



Case B: Regular and Irregular Masonry type, transversely not connected

Double – leaf wall made by an internal part with regular, squared bricks and the external side with mixed stone structures. There aren't enough elements that guarantee a transverse connection between the two wall face and instability may occur.



Case C: Irregular Masonry type, transversely not connected

Double – leaf wall composed by a mixed stone/brick structures, with a chaotic internal composition. Total absence of elements that guarantee the transverse connection and the instability occurs due to static or dynamic actions.

Two flat jack tests, carried out in the early 2000s in two SUs (SA25/SU199 and SA25/SU200), provide the order of magnitude of the mechanical properties of the masonry typology analysed (Table 3.4) (Cardani 2004). These values are similar to the “Disorganised stone masonry”, proposed by the current Italian code (NTC 2018) and its application rule (C.M. n.7 2019) (Table 3. 5). In this sense, the mechanical properties chosen for all SUs of the case study are those of the code, modified with the confidence factor 1.2, after Table 3.1 of the (EN 1998-3:2005).

Table 3.4: Results of the test conducted by flat jacks in AS25_US200 and AS25_US199

Parameters analysed	Units	SA25/SU200	SA25/SU199
Local stress	N/mm ²	0.07	0.15
Max stress applied	N/mm ²	0.76	0.81
Young Modulus (0.07–0.40 N/mm ²)	N/mm ²	605	728
Poisson ratio (0.07–0.40 N/mm ²)	-	0.02	0.21

Table 3. 5: Mechanical Parameters for a disorganised stone masonry following Table C8.5.I of (C.M. n.7, 2019)

Masonry typology	w [kN/m³] weight	f _m [N/mm²] Compressive Strength	τ ₀ [N/mm²] Shear Strength	E _m [N/mm²] Young Modulus	G _m [N/mm²] Shear Modulus	
Disorganised stone masonry	19	Min	1.0	0.018	690	230
		Max	2.0	0.032	1050	350
		Average	1.5	0.025	870	290

3.3.4. Retrofitting systems

The historical development of masonry aggregates strongly influences their seismic vulnerability assessment through statistical methods; this aspect is even more relevant if poor information about the buildings is available. For example, by comparing the retrofit year with the technical standards of that time, it is possible to have an idea of the restoration techniques employed and, therefore, of the possible related vulnerabilities (Centroni and

Filetici 2011). An investigation of the construction year of the buildings and of the eventual application of retrofitting technique was performed, after gathering and analysing original documentation and drawings. In general, masonry buildings constituting SAs were realized before 1900, except for SU1019, built after 2000, and SUs 95, 96, 120, 170 and 173, reconstructed in the 1980s (Figure 3.6).

Retrofit operations detected in Campi Alto di Norcia can be divided into two main categories in relation to their execution, i.e. before or after the Valnerina earthquake of the 1979 (*old/modern retrofit*). The most common operations, in the first case, were the addition of masonry buttress to the external wall of the SU, the introduction of metal ties to connect the opposite wall facades of the buildings, etc. Modern retrofit – performed even after the 1997 Umbria-Marche seismic event, consisted in the replacement of timber roofs and timber floors with concrete slabs, in the application of injections and steel stitching to fill the voids in the masonry walls and to connect different masonry faces. Concrete plates to connect the walls with the new floors and roofs are also common, as well reinforced cement-based plaster to reduce the irregularity of the masonry composition.

Figure 3.12a shows the percentage of the buildings retrofitted and constructed before and after the 1982. In this year the recovery plan for Campi Alto di Norcia was issued, which determined the procedures and guidelines for the reconstruction (P.D.R. 1982). In Figure 3.12b the distribution of the retrofit within Campi Alto di Norcia is highlighted in relation to the year of the projects, according to the documentation collected by the Umbrian Region. A comparison between the year of the retrofits and the construction code in force at the time, can provide an indication of the type of techniques adopted.

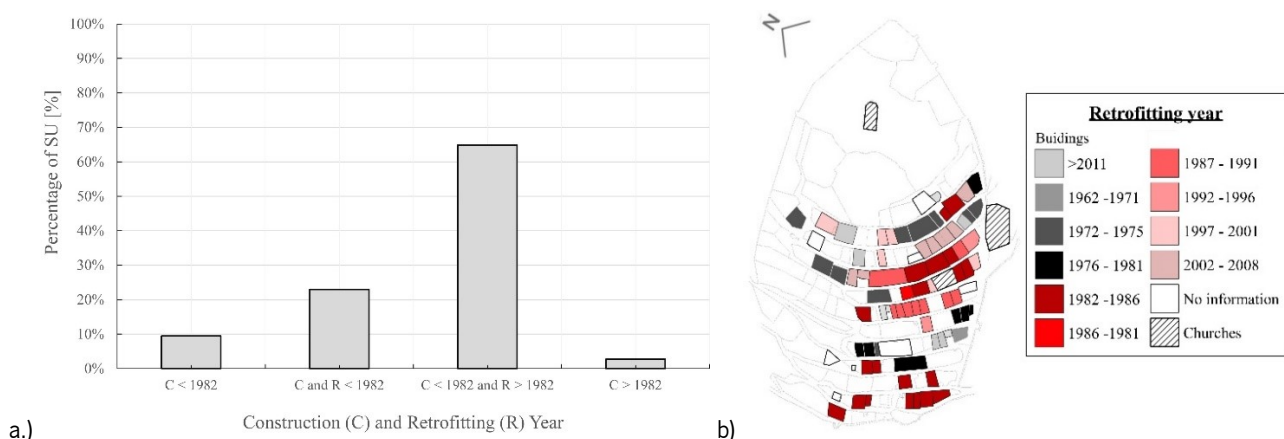


Figure 3.12: Percentage of the building stock analysed in relation to the construction and retrofitting year (a). An example of the survey sheets, adopted in this phase, is provided as an important output of the thesis in Annex A.

All the retrofits detected for each building of Campi Alto di Norcia are summarized in the output of the thesis “Annex B”: while the interventions detected during the inspections are highlighted in red, the black crosses define the retrofits evaluated through documentation. Figure 3.13 shows the distribution of the retrofit typologies in the case study.

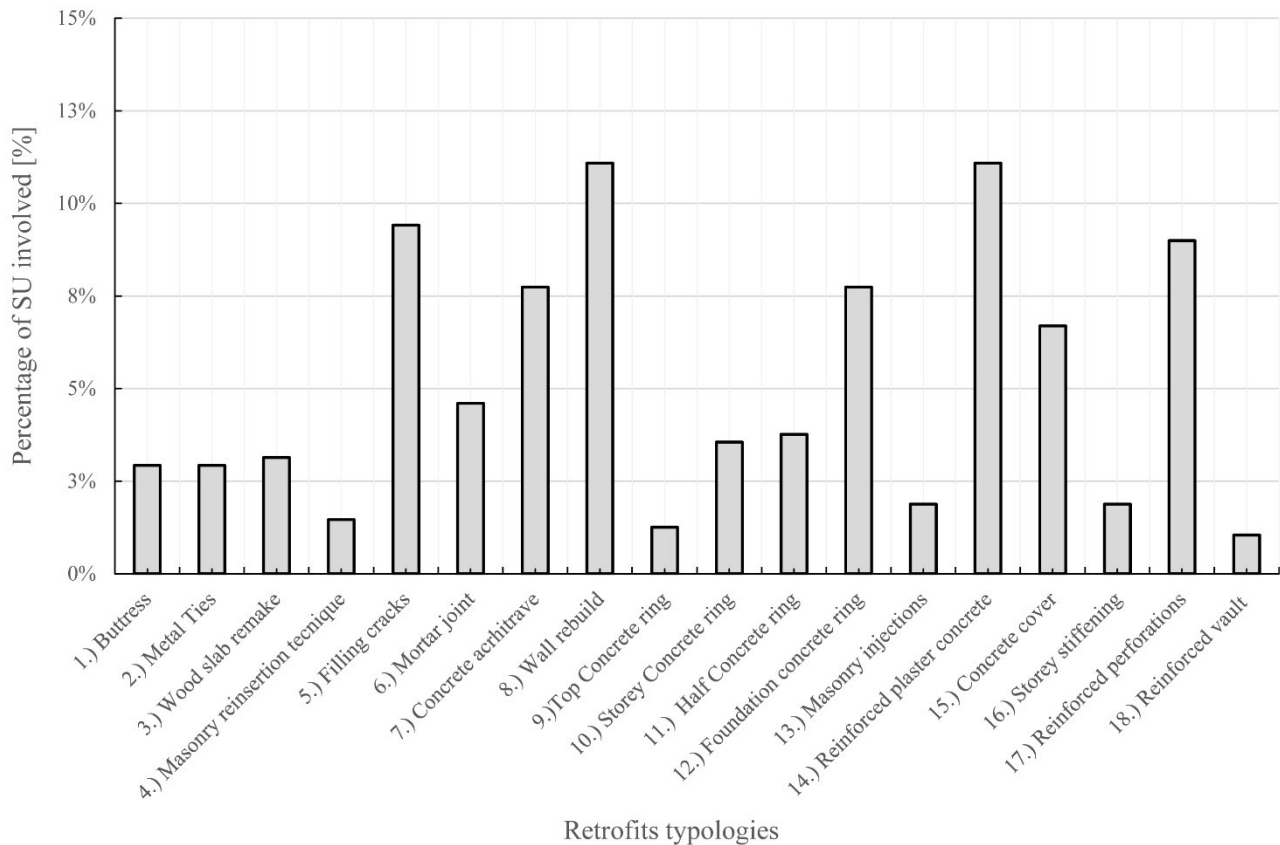


Figure 3.13: Retrofits typologies detected in the buildings of Campi Alto di Norcia

3.4. Conclusions

The knowledge phase of Campi Alto di Norcia, hit by the 2016 earthquake, is a valuable opportunity to analyse the urban organization of a typical medieval historical centre, as well as the dynamic behaviour of its masonry aggregates. Moreover, this case study is an example of a construction typology, widespread in Central Italy.

The methodology used for the knowledge phase is developed on the basis of the experience gained in other historical centres, studied in the previous phase of the research; the standardization of the knowledge activities proved to be very useful for the analysis of masonry aggregates, which are the result of an evolution process occurred during the centuries.

Many information are collected, using several tools (historical and archival documentation, previous research activities, site inspections, etc.) and shown as output of the thesis (*Annex A* and *B*). In this sense, the GIS environment has proved to be a powerful system for data storage and management, providing also cross correlation analysis between building characteristics and damage detected.

The architectural shape of the buildings follows the morphology of the ground: SAs are built on three different levels, orthogonal to the slope of the hill. While the south façade can reach three levels (max 12m), the north façade has in average one floor (max 4m). The percentages of openings in the façades are minimal and are irregularly distributed among the adjacent SUs. In general, the structures are made by double – leaf walls, formed by irregular masonry typologies, with poor mechanical properties, and bad connected in the transverse direction.

The high seismic hazard of the territory strongly influences the history of the urban development and the structural features of the buildings which are often rebuilt or retrofitted. Most of the latter detected in the case study are carried out following the 1979 earthquake and involve the introduction of techniques widely developed at the time: floor and roofs, made by timber structures, are replaced by heavy rigid slabs that are bad or good connected with perimeter walls.

As a general remark, the fragilities detected in buildings are caused by the inconsistencies between the reconstructed and preserved parts, as well as the inadequate the connections between the new and the original masonry. An example is the cement-based plaster employed for the reconstruction of the upper floors, just added for aesthetic reasons. Similarly, the demolition and reconstruction of vertical and horizontal structural elements with modern materials at the ground floor level result in local vulnerabilities (Borri et al. 2018).

3.5. References

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CHAPTER 4. Central Italy earthquake of 2016 and 2017

The chapter highlights the seismic history of the territory where Campi Alto di Norcia is located, as well as the damage detected in the case study following the 2016 earthquake. The information is collected through site inspections and archival documentation. The data are post processed, according to the macro seismic approach, with the EMS - 98 scale, and the results are used as the base to calibrate the Improved seismic vulnerability method and the new seismic risk procedure.

Chapter outline

4.1 Introduction

4.2 Seismic history of Valnerina valley

4.3 Characteristics of the 2016 earthquake

4.4 Damage detected in Campi Alto di Norcia

4.5 Conclusions

4.6 References

4.1. Introduction

As described in the previous chapters, a historical centre is the result of an evolution occurred during the centuries, defined by the structural alterations and superposition of different materials. Moreover, earthquakes produce nonlinear time histories acceleration of ground motions. For this reason, when a strongly irregular dynamic action hits an old building stock composed by masonry structures, the likely effect is a chaotic damage. The purpose of this chapter is to analyse the latter and to use it for later calibration. The study is carried out in a systematic way, starting from the analysis of the seismicity of the area, evaluating the dynamic characteristics of the earthquake, as well as organizing the damage detected during the inspections.

Historical documents and data collection allow to define the seismic history of Valnerina, the region where Campi di Norcia is located affected by the 2016 seismic events, as well as to understand its geological framework.

The Central Italy earthquake is then examined through the data obtained with the accelerograms recorded by the CAMPI station (CMI), installed at Campi Basso, to evaluate the shake intensity and the characteristics of the principal directions of the ground motion.

Finally, the seismic damage is detected in Campi Alto through site inspections, carried out between October 2018 and Summer 2019, and the analyses of AeDES forms² (AA.VV. 2014), evaluated by technicians shortly after the earthquake. The macroseismic intensity scale measures the effects of an earthquake at a given location and is evaluated through the damage detected. In this sense, an official report (Galli et al. 2017) provides an intensity of I_{MCS} equal to 8.5, evaluated with the MCS scale (Musson and Cčić 2011) for the seismic events

² The AeDES forms allow to collect information, in a uniform and quick way, about the type and amount of damages on structural and non-structural members of buildings hit by a seismic event. The purpose is to distinguish the buildings that can be used from those that must be totally or partially forbidden.

involving Campi Alto di Norcia during the 2016. This means that the damage of the case study appears great in poorly built structures and considerable in specially designed structures: fall of chimneys, factory stacks, columns, monuments, walls, heavy furniture overturned and buildings shifted off foundations. The failures of masonry aggregates are organized and divided between local mechanisms and global behaviour and then classified considering the EMS-98 damage scale.

4.2. Seismic history of Valnerina valley

The geological framework of Valnerina valley is strictly connected to the evolution process of the central Apennines, characterized by a compression of the ground that generate the current mountain ranges, following a relaxation phase, which caused different valleys and geological faults. The earthquakes that hit this area testify these geodynamic processes which led to the shortening of the Apennines fold and thrust system are still active (Console et al. 2018). The DISS Working Group (2015) plots the surface projection of the presumed extent of the Apennines fault, basing on the past earthquake events. Figure 4.1a shows the surface projection of the presumed extent of the fault at seismogenic depth, according to the largest earthquakes reported in Guidoboni et al. (2018). The sources encircled in yellow delineate the system of large active extensional faults running on top of the Apennines. This system is responsible for the majority of the earthquakes occurring in the Italian peninsula (Valensise et al. 2017). On the base of this seismogenic sources, seismic hazard maps are plotted, in terms of PGA (g) (Zonno et al. 2012). The latter is identified by the Italian seismic classification (D.P.R. 380 2001), that divides Italian territory into four seismic zones, with different acceleration intervals, having 10% probability of exceeding in 50 years. The Valnerina region is located in the first seismic zone: the most dangerous one, where the probability of earthquakes with a PGA greater than 0.25g is high (Figure 4.1b).

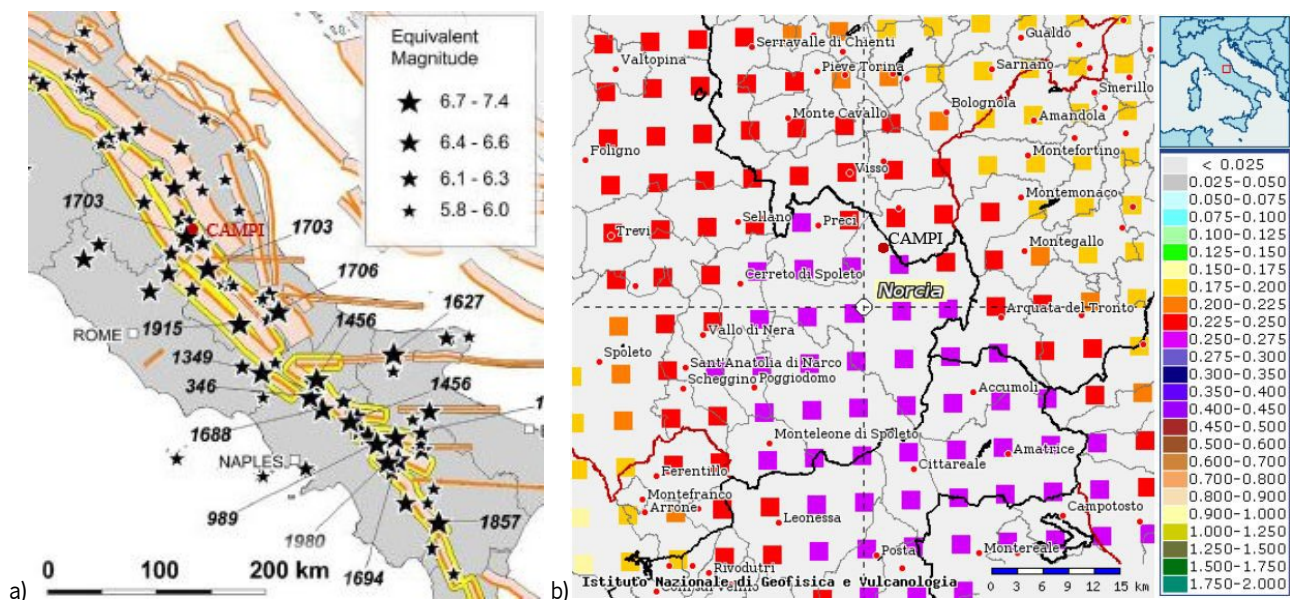


Figure 4.1: Composite Seismogenic Sources from the DISS database (adapted from Valensise et al. (2017)) (a); Seismic Hazard Map of the Valnerina Region, expressed in terms of PGA, where the city of Norcia and Campi Alto are highlighted (b).

The main earthquakes that hit this region and the case study of Campi Alto di Norcia are selected from the database of Guidoboni et al. (2018) and are shown in Table 4.1 and Figure 4.2. The average seismic intensity of these events, evaluated with the MCS scale, is around VIII - IX, while the return period (recurrence) is 130 years for severe earthquakes (VIII) and 345 years for violent ones (IX).

Table 4.1: Main earthquakes that hit the Valnerina region in the history (source: Guidoboni et al. (2018)).

Date (yyyy-mm-dd)	Epicentral Area	N° of Macroseismic Observations (NMO)	Epicentral intensity - MCS scale (I _e)	Equivalent magnitude based on macroseismic observations (M _s)	Seismic Intensity (MCS) in Campi Alto
63 B. C. --	Spoletto (PG)	1	8	5.6	-
99 B. C. --	Norcia (PG)	2	8	5.6	-
127 04 30	Apennines	16	9	6.5	-
1328 12 04	Valnerina	13	10	6.4	10
1477 02 03	Foligno (PG)	3	6.5	4.9	-
1599 11 06	Valnerina	20	9	6	9
1639 10 08	Laga Mount	17	10	6.1	-
1703 01 14	Apennines	197	11	6.7	11
1719 06 27	Valnerina	16	8	5.5	8
1730 05 12	Valnerina	115	9	5.9	9
1791 10 11	Apennines	54	8	5.5	-
1815 09 03	Valnerina	24	8	5.5	-
1838 02 14	Valnerina	24	8	5.3	-
1859 08 22	Valnerina	18	9	5.8	9
1898 08 25	Valnerina	67	7	5	-
1910 06 29	Valnerina	58	7	4.9	-
1974 12 02	Valnerina	27	7.5	5.2	-
1979 09 19	Valnerina	694	8.5	5.8	8.5
1997 09 26	Apennines	891	8	5.7	6

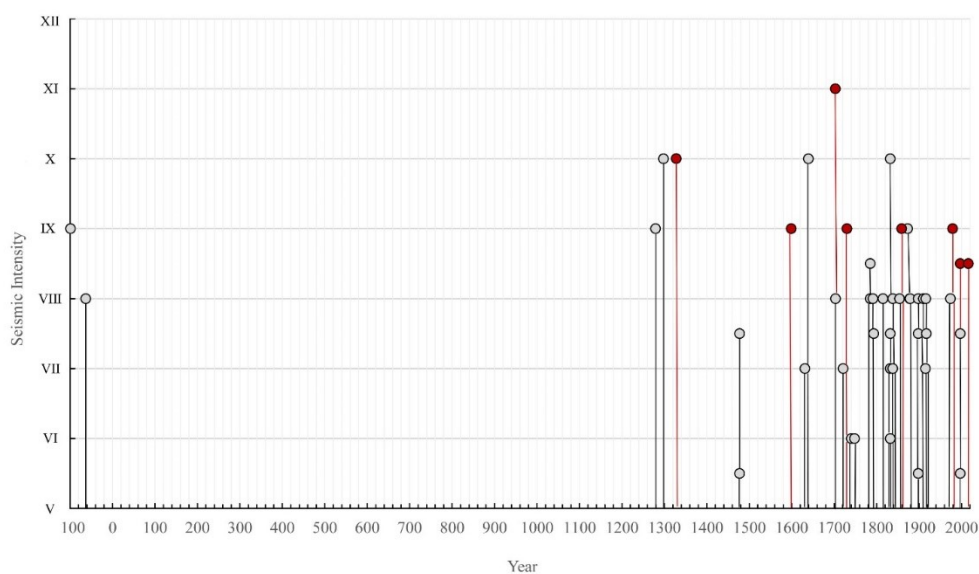


Figure 4.2: Timeline showing the seismic history of Valnerina region, based on the data of Table 4.1, where the earthquakes that hit Campi Alto di Norcia are highlighted in red.

The information about the earthquakes that hit Campi Alto are connected to the damages of the nearby settlement of Norcia, being it a larger urban center of greater historical importance. The seismic events that caused the greatest damage to the case study occurred in 1328, 1599, 1703, 1730, 1859, 1979 and 1997, (Monachesi 1987), (Boschi 1998), (Bertolaso and Boschi 2007), (INGV 2019).

1328

During the earthquake of 1328 in Norcia almost all the buildings and the perimeter walls of the city collapsed; many settlements around Norcia suffered a lot of damages and several deaths, even if there are no reliable data on the condition of the case study.

1599

Before the main shock, many vibrations caused cracks to different buildings in Norcia. After November 6th, 40 buildings collapsed throughout the Norcia area and there were different casualties. There isn't accurate information about the damages caused in Campi Alto.

1703

The earthquakes of January-February 1703 are the most significant seismic period of the last millennium in Central Italy: cracks opened in the nearby mountains, human and material losses registered in all the Region including the L'Aquila territory and collapse of some arches of Colosseum in Rome (Valesio. F. 1977). In Norcia the earthquake triggered a strong economic crisis, due to the deaths and the destruction of the productive activities, which caused a strong migration from the region. In Campi Alto twenty houses collapsed, while others twenty-five became unusable; one church suffered repairable damage, while two other churches were ruined; the perimeter walls of the castle collapsed and two towers felt, but no casualties were registered (Cardani 2003). Engravings detected on the stones of some masonry buildings of Campi Alto di Norcia testify the events occurred (Figure 4.3).

1730

In Norcia, the bell tower of San Benedetto collapsed on the buildings below and all the churches and the public buildings were severely damaged. the Prefect's home and the Consular Palace. The region was exempted for 10 years from paying taxes (Pirri 1960). In Campi Alto, the earthquake destroyed most of the houses and caused casualties The main church of the settlement, Saint Andrea, was restored after the earthquake (Garibaldi 2018)

1859

Half of the buildings stock in Norcia collapsed, while the other constructions registered several damages or partial collapses on the upper floors. Almost all the churches were hurt, as well as the public buildings. The neighbouring settlements around Norcia, including Campi Alto were damage. The difference between the Gregorian Cadastre (AA.VV. 1820) and the map of Campi Alto (AA.VV. 1939) shows the damages occurred and several retrofits of that period were detected during the inspections, such as masonry buttress built at the end of the aggregates or attached to the main façades of the buildings.

1979

The seismic event hit the mountain settlements, damaging the historical - cultural heritage. In Campi Alto the damage was limited and intensified by the condition of the buildings: cracks of previous earthquakes, masonry decay, buildings abandoned and in a state of ruin (Cordella 1981). According to the recovery plan different buildings were rebuild and retrofitted through the replacement of timber roofs and timber floors with concrete slabs, the connection of the walls with the new floors and roofs with concrete plates, as well as the reinforcement

of the façades with cement-based plaster to reduce the irregularity of the masonry composition. (Poggiolini 1982).

1997

The earthquakes between September and November 1997 hit the region of the Apennines Umbria-Marche, damaging in different periods several buildings and churches, including the vault of Giotto in the Basilica Superiore of San Francesco, in Assisi. The material losses were considerable, because of the large area of territory involved, and there were less casualties (Camassi et al. 1997), (Fazio and Bondesan 2007). The damage detected showed the low level of effectiveness of the 1979 retrofits in relation to the seismic capacity of the buildings, which was decreased. Less damages were registered in Campi Alto, such as widespread cracks in the masonry walls and detachment some roofs from the vertical structural elements.

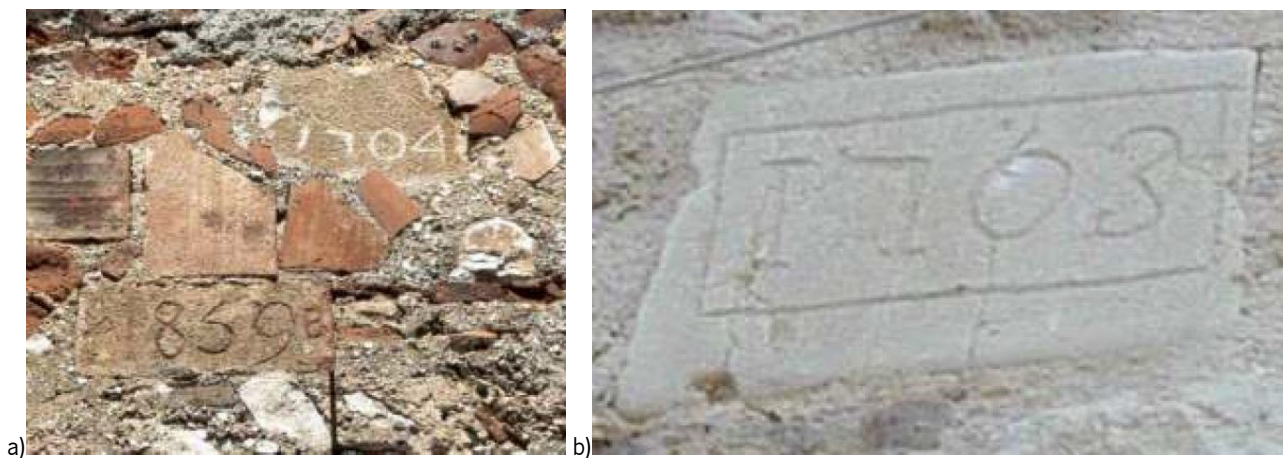


Figure 4.3: Engravings detected in Campi Alto di Norcia (SU_160 and SU_E) that report the year in which the earthquakes occurred (Cardani 2003).

4.3. Characteristics of 2016 – 2017 earthquake

The 2016 earthquake in Italy consists of several seismic events that affected a large area of territory located along the central Apennines, between the Region of Lazio (L), Umbria (U), Marche (M) and Abruzzo (A), from Summer 2016 to January 2017. The first strong earthquake occurred on 24 August ($M_w = 6$), with the epicentre positioned in the Tronto Valley, between the historical centres of Arquata del Tronto (M) and Accumoli (L) (INGV 2016a). The latter with the near village Amatrice were completely destroyed, causing hundreds of casualties, injured and homeless. During earthquake swarms, two strong aftershocks took place: the first on 26 October ($M_w = 5.9$) with epicentres on the Umbria-Marche borders, between the villages of the province of Macerata di Visso (M), Ussita (M) and Castelsantangelo sul Nera (M), while a second on 30 October ($M_w = 6.5$) with epicentre between the cities of Norcia (U) and Preci (U) (INGV 2016b). This last event caused the collapse of several buildings and above all churches in a wide area (Garibaldi 2018), such as the basilica of San Benedetto in Norcia and the church of San Salvatore in Campi Basso. The events of October 26 and 30 didn't cause any casualties due to the fact that residents were evacuated from damaged and unsuitability houses after the previous seismic events. Other aftershocks took place in January 2017, but had a lower magnitude. Figure 4.4

and Table 4.2 show the evolution of the 2016 –2017 earthquake sequence, identifying the position of all the main shocks and the epicentre areas.

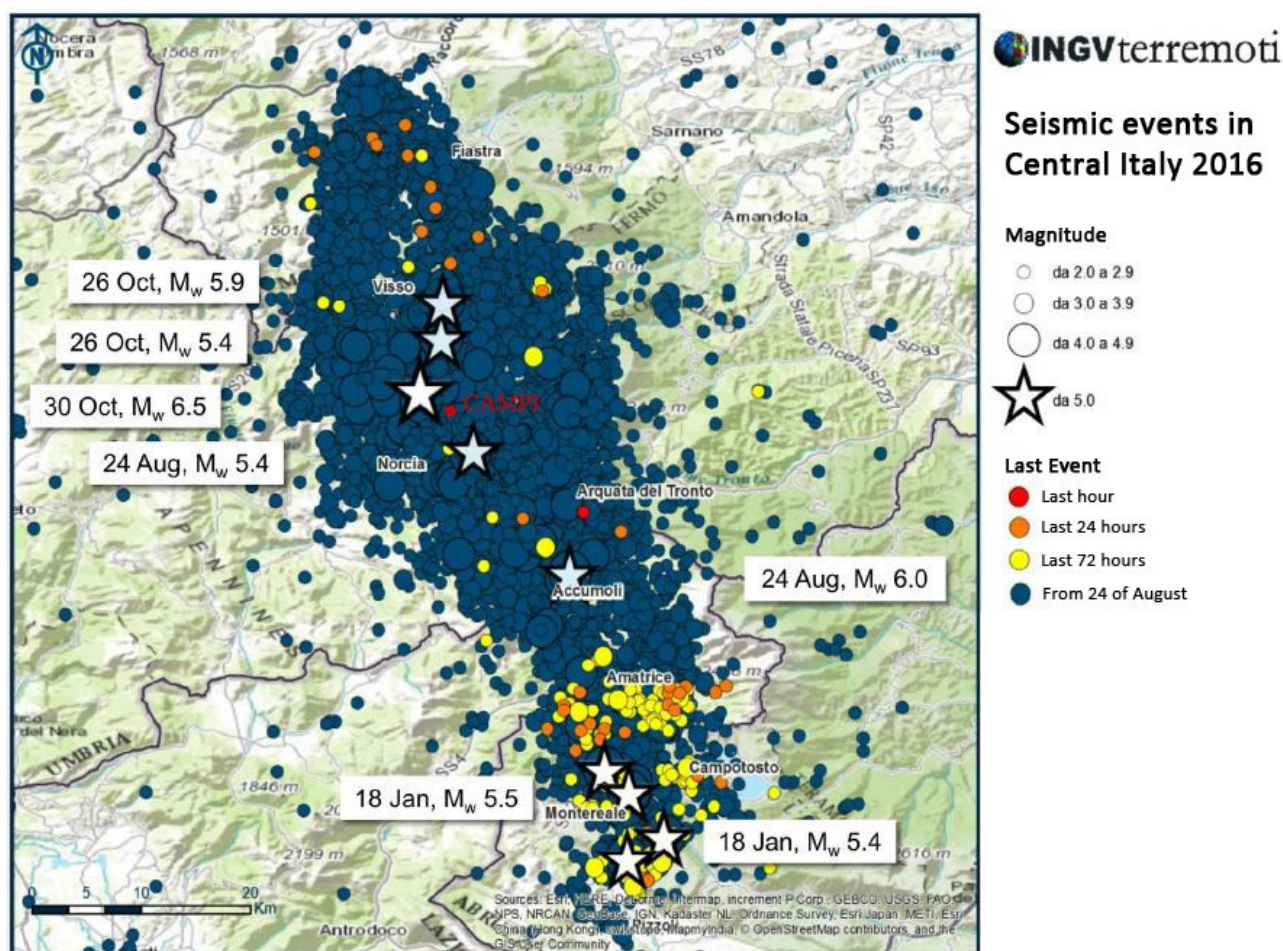


Figure 4.4: The map of the seismic swarm in Central Italy, from 24 August 2016 to 20 January 2017. The last events are recorded on the 23 of January 2017, where the main seismic activity is concentrated in the south area between the provinces of L'Aquila and Rieti (adapted from Valensise et al. (2017)).

Table 4.2: Parameters of the main seismic events of the earthquake swarm.

Earthquake date (yyyy – mm – dd)	Origin time (UTC)	Magnitude (M _w)	Epicentre Area	Hypocentral Depth (Km)	Latitude (N°)	Longitude (E°)
2016 08 24	01:36:32	6.0	Accumoli (L)	8.0	42.70	13.23
2016 08 24	02:33:28	5.4	Norcia (U)	7.5	42.79	13.15
2016 10 26	17:10:36	5.4	Visso (M)	8.7	42.88	13.13
2016 10 26	19:18:05	5.9	Visso (M)	8.0	42.91	13.13
2016 10 30	06:40:17	6.5	Norcia (U)	9.0	42.83	13.11
2017 01 18	10:14:09	5.5	Montereale (A)	10.0	42.53	13.28
2017 01 18	10:25:23	5.4	Montereale (A)	9.0	42.49	13.31

The greatest damage registered in Campi Alto di Norcia was caused by the earthquakes of October 26 and 30. The accelerograms recorded by the CAMPI station (CMI), installed at Campi Basso (latitude 42.85;

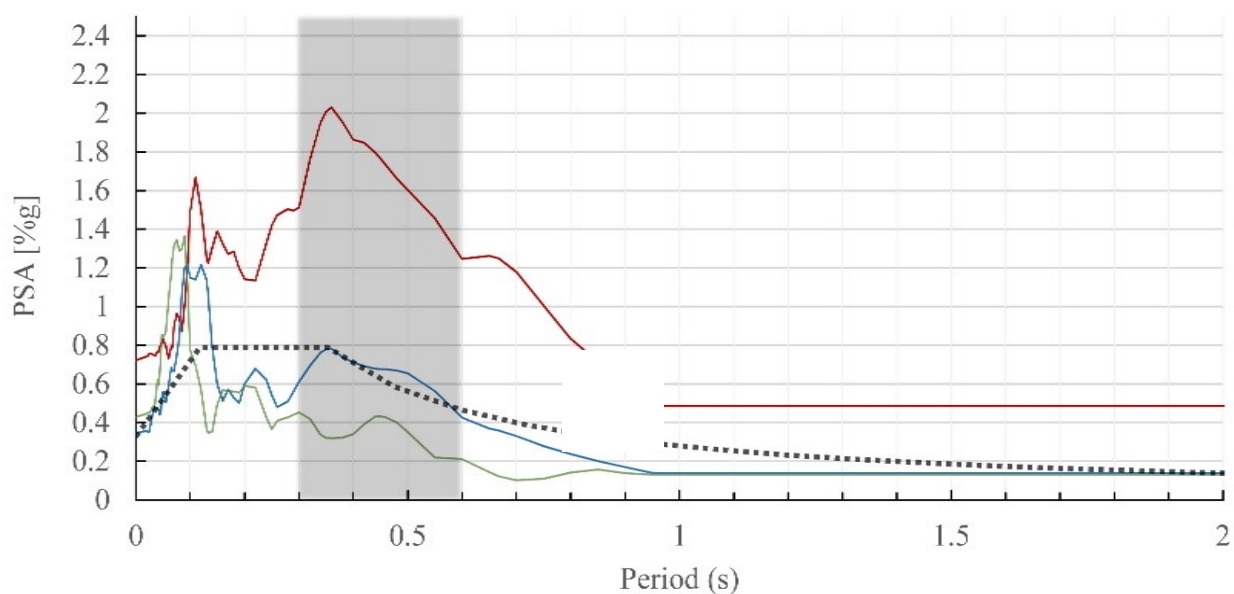
longitude:13.09; altitude: 767 m a.s.l.), provide a clear overview about the seismic input that hit the case study (Luzi et al. 2008), (Pacor et al. 2011).

The main shocks, occurred between 26 and 31 October, are extrapolated from the strong ground motion of the earthquake and post processed. The seismic input is defined in terms of peak ground acceleration (PGA) and is expressed as a percentage of gravitational acceleration [%g], considering the two horizontal components (East – West and Nord - Sud direction) and the vertical component (Up – Down direction). In this sense, it is possible to compare the values obtained with the seismic hazard evaluated for this zone from the Italian code, characterized by a PGA greater than 0.25g (Seismic Zone 1).

Table 4.3 shows the maximum PGA registered by the Campi station for the different seismic events analysed. Even if it isn't correct to compare single PGAs only with forecasted shakings (PGA with 10% probability of exceedance in 50 years), since probabilistic models cannot be validated (or rejected) on the basis of a single event, this process is useful to understand the consistency of the model suggested by the Italian Code (Meletti et al. 2016). In this sense, the maximum PGA is sensitive higher than the one generally used for the structural analyses of the buildings located in this area and, therefore, provide an important information about the action to which the structures were subjected and the resulting damage (Figure 4.5).

Table 4.3: Max PGA (%g) recorded by the accelerogram of the Campi Station in the three main directions for different seismic events

Earthquake date (yyyy – mm – dd)	Origin time (UTC)	Magnitude [Mw]	East – West [%g]	Nord – Sud [%g]	Up – Down [%g]
2016 10 26	17:10:36	5.4	0.713	0.341	-0.429
2016 10 26	19:18:06	5.9	0.644	0.308	0.489
2016 10 26	21:42:01	4.5	0.212	0.207	0.115
2016 10 31	07:05:45	4.0	0.180	0.087	-0.083



a) — East - West — Nord - Sud — Up - Down Italian Code

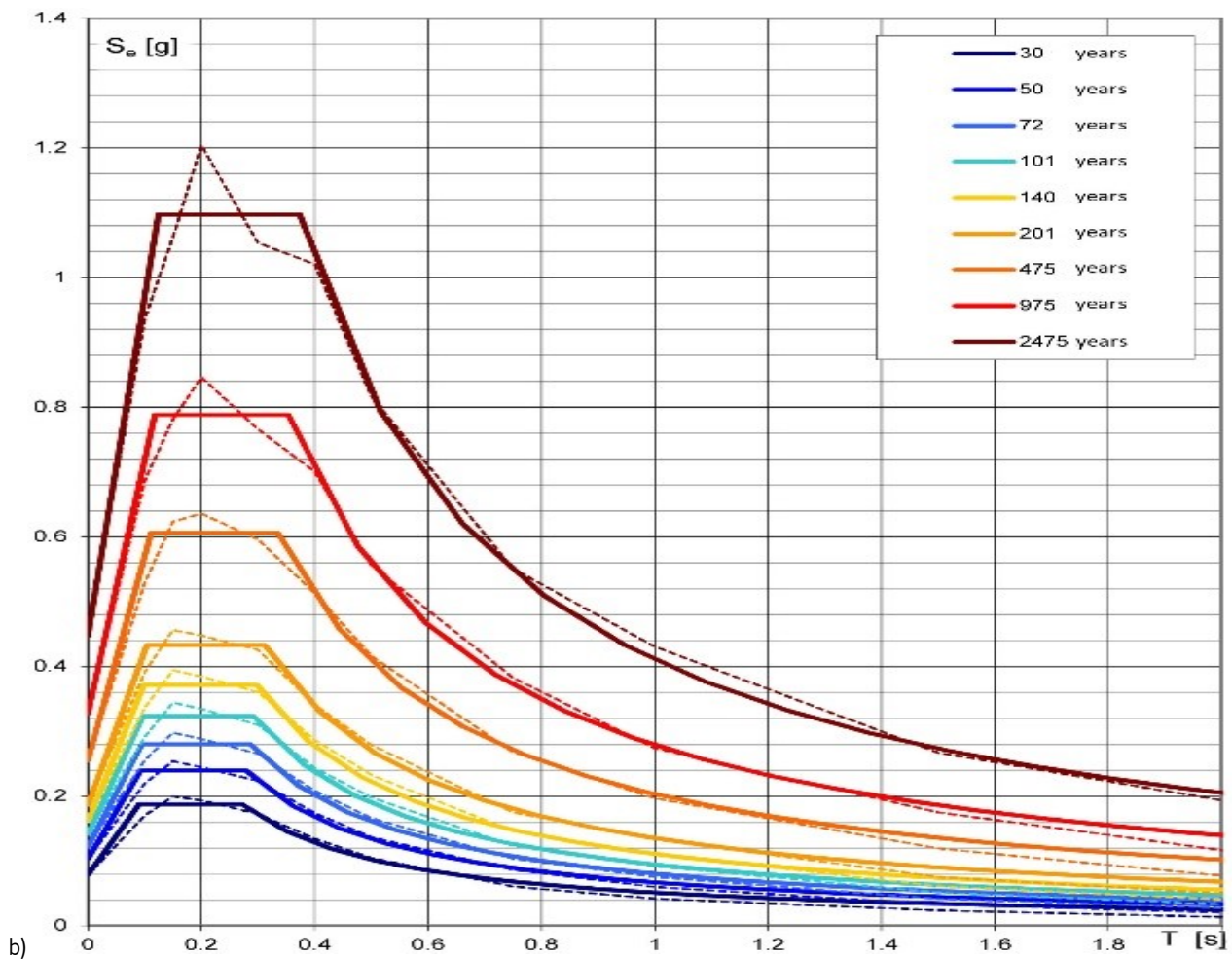


Figure 4.5: Comparison of the Spectras in three directions evaluated with the accelerograms of the earthquake occurred on 26.10.2016 at 19:10:36 and the spectrum defined by the Italian code for the seismic zone of Campi considering the collapsed limit state. The range of the probable first period of masonry aggregates is highlighted in grey (a); Elastic response spectrum defined for different return periods of Campi Alto di Norcia with the electronic spreadsheet developed by Prof. Braga and Ing. Lomiento, according to the Italian Code (<http://anidis.it>) (b).

Further evaluations can be performed by overlapping the seismic accelegrams of the three directions in a single diagram. Figure 4.6 shows a 7-second extract of the recorded ground shaking motion (30 - 40 seconds), in which the acceleration peaks are concentrated. In this sense, it is possible to evaluate the synchronous and asynchronous superimpositions of the accelerations in the three directions and the influence of combined horizontal and vertical components on the dynamic nonlinear response of masonry structures. The upward seismic acceleration, that acts almost simultaneously to the horizontal component, causes the reduction of the self-weight of the masonry and, therefore, a partial loss of friction, that provides a stabilizing effect to the structural system. The result is a more extensive failures in structures with small cohesion, whereas it has limited effects in structures with medium cohesion (Liberatore et al. 2019), as well as an unfavourable effect to the stability of non-structural elements (chimneys, etc.) (Breccolotti and Materazzi 2016). These analyses allow to better understand and provide a clear overview of the causes that triggered the damage detected in the case study, i.e. local mechanisms and disintegration of masonry buildings (Mariani 2018).

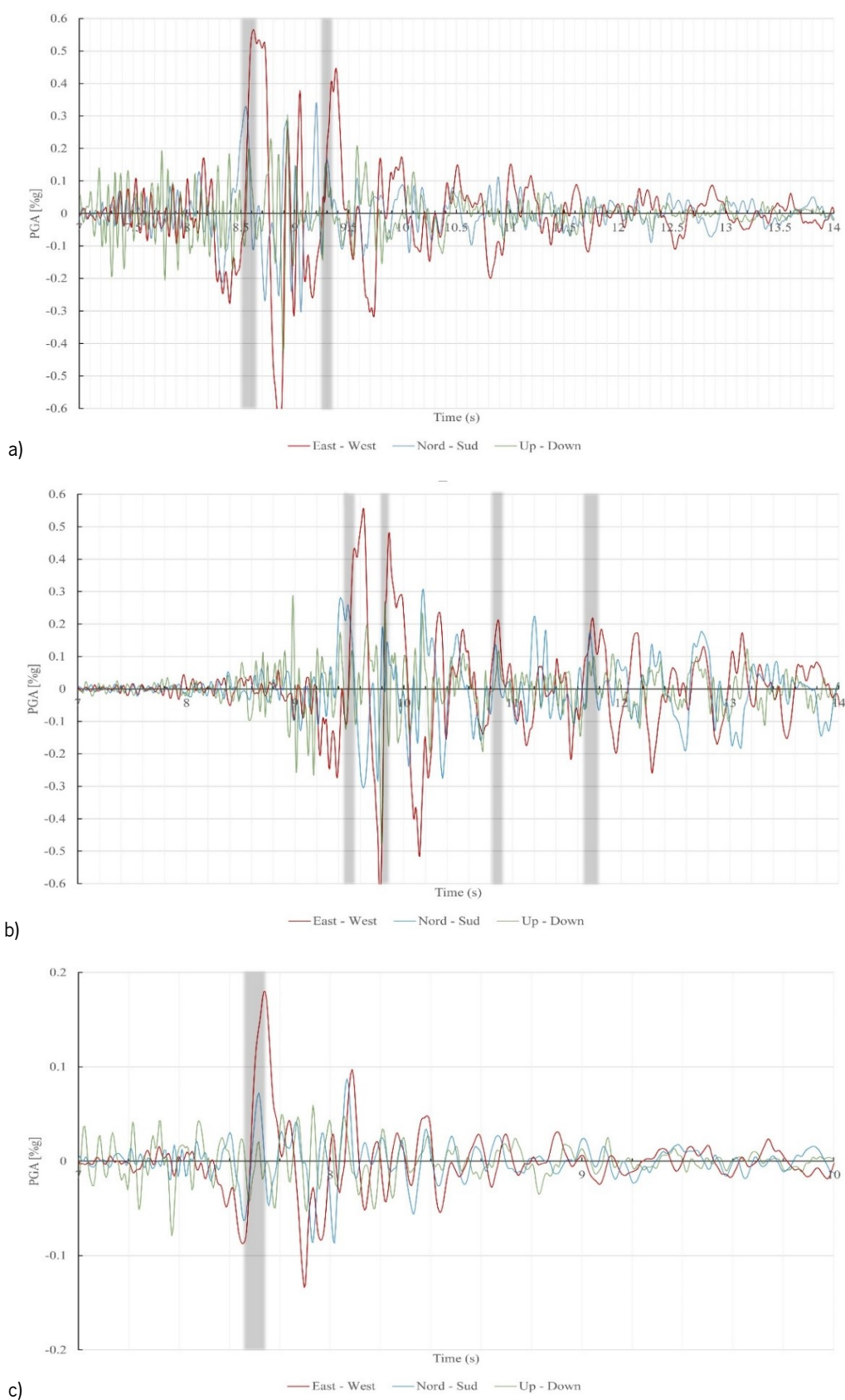


Figure 4.6: Accelerograms of the earthquake occurred on 26.10.2016 at 19:10:36 (a), at 21:18:06 (b) and on 31.10.2016 at 09:05:45 (c), highlighting in grey the period in which the PGAs over 0.2g are added.

4.4. Damage detected in Campi Alto di Norcia

The seismic event of August and October 2016 required the deep in-situ surveys of the structural damages undergone by the buildings constituting the historical city centre of Campi Alto di Norcia. Inspections were performed, in the immediate post-emergency phase, by expert technicians following the indications of the AEDES forms (National Civil Protection Department, 2014) for the detection of the partial and local collapses, cracking scenarios, structural failures, resulting deficiencies, etc (De Martino et al. 2015).

The inspections carried out revealed a sliding phenomenon toward the valley, of the hill where Campi Alto is located, shown by the damage occurred to the road system that connect the masonry aggregates along the slope of the hill and by a crack in the ground at the top of the historical centre, near the church Saint Maria della Neve (Figure 4.7). The walkways of Campi Alto were obtained by adding and levelling inconsistent material between two masonry buildings or above retaining walls. The sliding phenomenon towards the valley of the ground caused the detachment of the roads from the masonry buildings and their partial collapse, due to the overturning of the retaining walls.



Figure 4.7: Ground crack detected under the dense vegetation (a) and street collapse of Via del Cucolo due to the overturning mechanism of the retaining wall (b).

In Campi Alto the church of Saint Andrea (XIV century) and the attached mediaeval gate of the castle are heavily damaged (Figure 4.8), while the churches of Madonna della Neve (Figure 4.9) and of Saint Maria di Piazza (Figure 4.10), built in 1351 with several frescos inside, are completely collapsed. The current condition of these buildings is reported to give historical evidence of the damage occurred, but they are not analysed in detail, since the aim of the research is the vulnerability assessment of masonry aggregates.



Figure 4.8: Saint Andrea Church Tower secured after the 2016 earthquake (a), façade of the Church and medieval gate of the town after seismic event (b) and before (c).

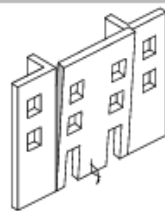
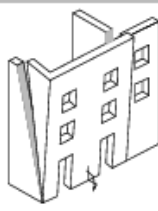
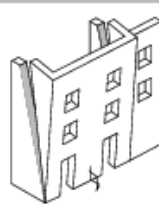
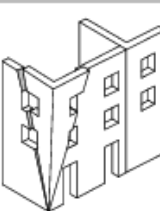
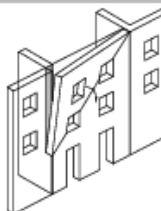


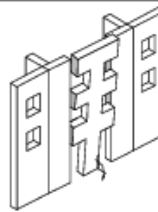
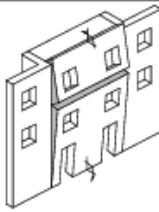
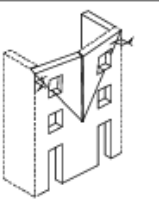
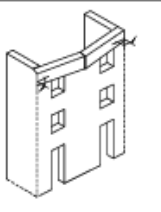
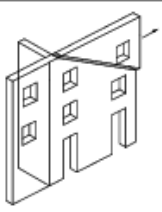
Figure 4.9: Madonna della Neve church after the earthquake 2016 (a) and before (b).



Figure 4.10: Saint Maria di Piazza church after the earthquake 2016 (a) and before outside (b) and inside.

The damage evaluated in Campi Alto is divided in relation to the seismic response of the masonry buildings, considering their in plane or out of plane behaviour. The failure mechanisms are, therefore, identified according to the abacus proposed by the FaMIVE method (Figure 4.11), in which local mechanisms are described through 8 different classes (from A to G). Furthermore, five classes are added to define the in plane failure (H), the partial failures (I, L), the roof/floors collapse and the masonry disintegration. The principal failure mechanisms detected are the vertical arch (25%) and the in plane failure (37%), while the lack of cohesion of the masonry walls of the buildings involved several masonry failures (12%). Roof/floors collapse occurs for the 6% of the SUs and corner failures, as well as horizontal arch, is detected for the 8% of the building stock.

A	B1	B2	C	D
VERTICAL OVERTURNING	OVERTURNING WITH 1 SIDE WING	OVERTURNING WITH 2 SIDE WINGS	CORNER FAILURE	PARTIAL OVERTURNING
				

E	F	G		H
VERTICAL STRIP OVERTURNING	VERTICAL ARCH	HORIZONTAL ARCH		IN PLANE FAILURE
		G	TOP STOREY	
				

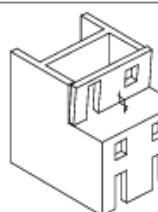
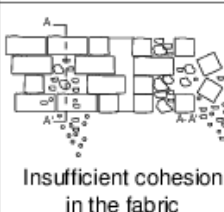
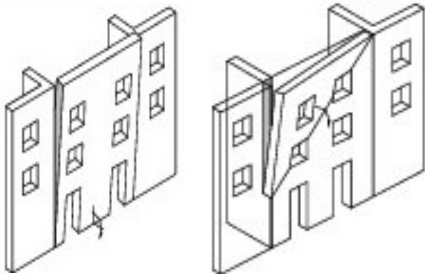
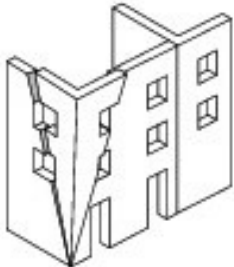
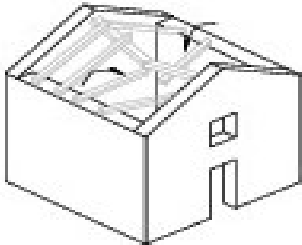
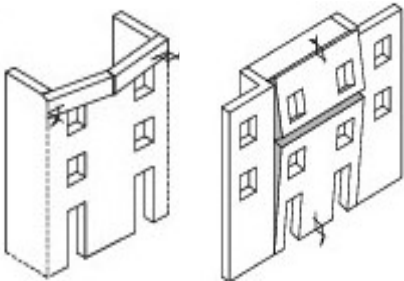
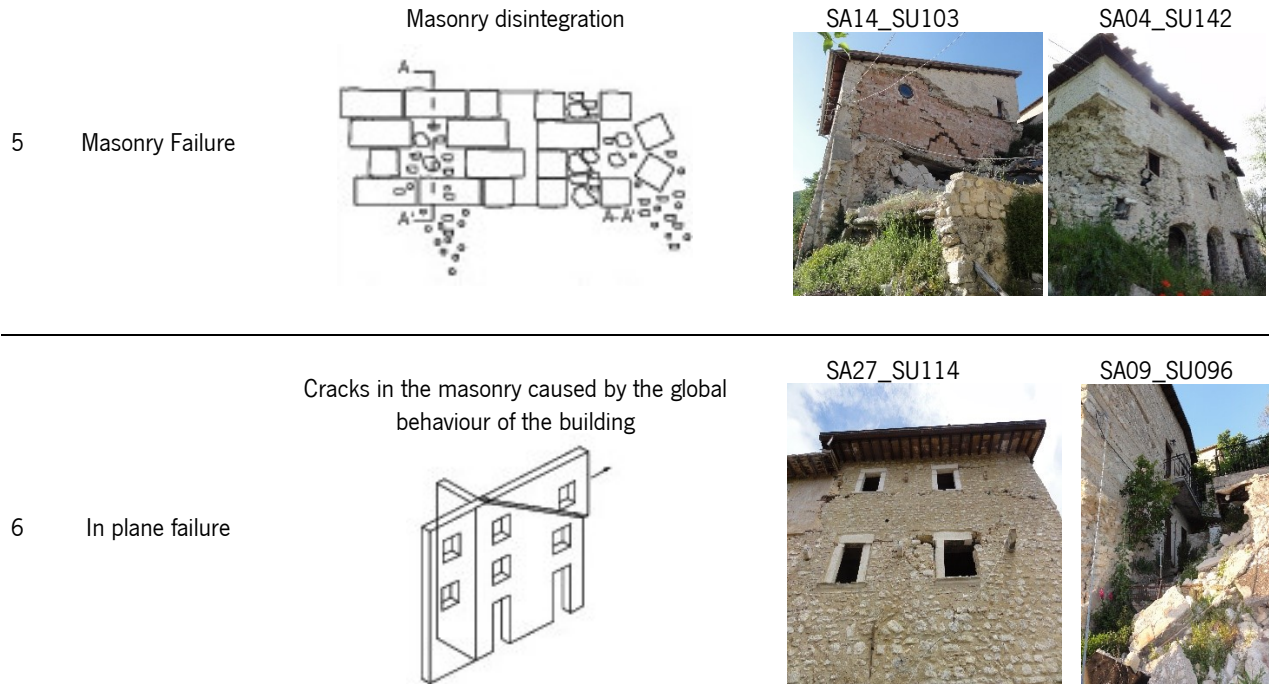
FURTHER PARTIAL FAILURES		ASSOCIATED FAILURES	
I	L		
VERTICAL ADDITION	GABLE OVERTURNING	ROOF/FLOORS COLLAPSE	MASONRY FAILURE
			 Insufficient cohesion in the fabric

Figure 4.11: Failure mechanisms considered by FaMIVE (D'Ayala and Speranza 2004)

Table 4.4: Example of the principal failure mechanism identified in the case study of Campi Alto di Norcia

Failure mechanisms	Short Description	Structural Units involved	
1	Vertical /Partial Overturning	SA21_SU183	SA12_SU165
	Overturning of the entire façade or big portions of the external walls		
			
2	Corner Failure	SA28_SU117	SA15_SU170
	Corner wall detachment defined by diagonal cracks		
			
3	Roof/Floors collapse	SA09_SU095	SA7_SU151
	Failure of the structural elements that form the horizontal system		
			
4	Horizontal / Vertical Arch	SA12_SU165	SA21_SU183
	Half overturning of the façade		
			



The different failure mechanisms surveyed are then reorganized to define for each building a specific damage typology, considering the simultaneous presence in the same building of local mechanisms and in plane failures and the case in which it is not possible to estimate the damage, as the building is intact or totally collapsed. In this sense, the term “Local” identifies a damage due only the activation of local mechanism and represent the 13% of the building stock analysed, while the “Global” one describes the in plane failure of the masonry structure (28%). The hybrid damage (24%) considers the simultaneous presence of the two type of damages in the building. The percentage of buildings which were not damaged are 21%, while on the contrary the 13% of the SUs completely collapsed.

Figure 4.12 summarizes the failure mechanisms according to the FAMIVE method and the general overview of the damage typologies detected in Campi Alto.

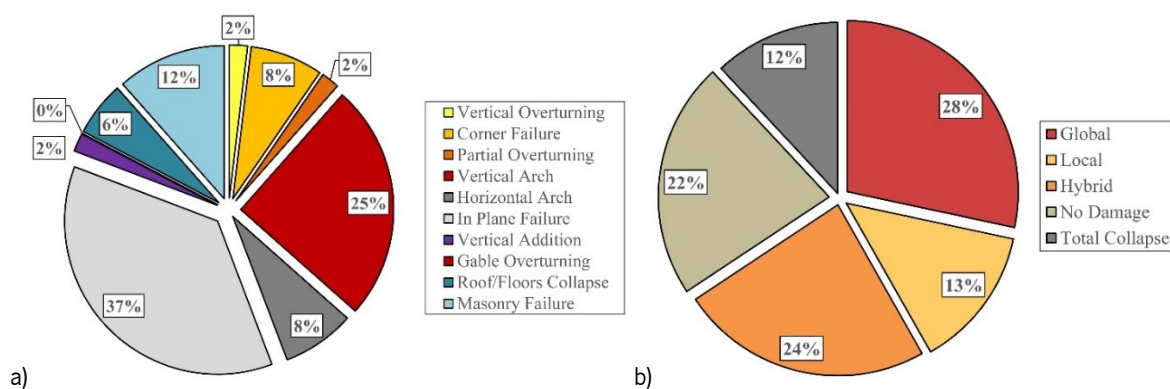


Figure 4.12: Failure mechanisms (a) and damage typologies (b) surveyed during the inspections of the masonry buildings.

The EMS98 scale (Grünthal 1998) is then adopted and applied to the different constructions with the aim of assigning a 'damage class' to each SU of Campi Alto di Norcia. As previous described in chapter two, the EMS98 scale provides graphical illustrations and descriptions of six different increasing Damage Levels - DL (i.e. D0, D1, D2, D3, D4 and D5 – corresponding, respectively, to the lack of damage, negligible to slight, moderate, substantial to heavy and very heavy damage, and full destruction). In the following images (Figure 4.13, Figure 4.14, Figure 4.15, Figure 4.16, Figure 4.17) are shown the assignment criterion of the classes of the buildings, damaged by the earthquake, using the five degrees of the EMS-98 scale.



Figure 4.13: Damage Class "D0-D1" assigned to the SU 197 according to the Gregorian Cadastre.



Figure 4.14: Damage Class "D2" assigned to the SU 1019 according to the Gregorian Cadastre.



Figure 4.15: Damage Class “D3” assigned to the SU 114 according to the Gregorian Cadastre.



Figure 4.16: Damage Class “D4” assigned to the SU 165 according to the Gregorian Cadastre.



Figure 4.17: Damage Class “D5” assigned to the SU 54 according to the Gregorian Cadastre (a) and SU 54 before the earthquake (b).

During the inspections emerged the importance to carry out the survey even inside the buildings to understand the level of damage, as the buildings are made by double leaf masonry walls. Figure 4.18 shows significant internal damages detected during the inspections executed with the Department of Architecture of the University of Ferrara, not visible from the outside, which bring the SU to a worse level of damage (D2 → D3) according to the EMS – 98 scale.



Figure 4.18: Wide cracks in the inner face of the walls inside the SU 117.

With the aim of reducing the variability related to the personal expertise of the surveyor (Baggio et al. 2010), three different independent judgements were used, applying then the average to the performed estimations. Figure 4.19 and Table 4.5 show the distribution of the damage level for SUs, in terms of percentage of buildings and volume of construction. Most SUs show a DL equal to D2-D3 and D4-D5, but most of the volume turns out to be in the range D2-D3, since many SUs, used as storage areas and cellars with a low volume, are nowadays fully collapsed. Storage areas and cellars were not further considered in the assessment procedure, therefore reducing the number of analysed SUs to (only) 67.

Table 4.5: Damage distribution organised according to the volume and the number of buildings involved.

Damage Level (DL) and general description		N° of SUs	% SUs	m³	%
D0	No damages	0	0%	0	0%
D0 - D1	No structural damage and slight non-structural damage	11	16%	2120	15%
D1 - D2	No structural damage and roof tiles detachment	9	13%	1550	11%
D2 - D3	Slight structural damage and partial collapse of chimney and roof tiles' detachment	25	39%	5260	38%
D3 - D4	Large and extensive cracks in most walls and chimney fracture at the roofline	9	13%	2030	15%
D4 - D5	Partial structure failure of roof and floors	6	9%	1510	11%
D5	Total collapse of the building	7	10%	1400	10%
Total		67	100%	13870	100%

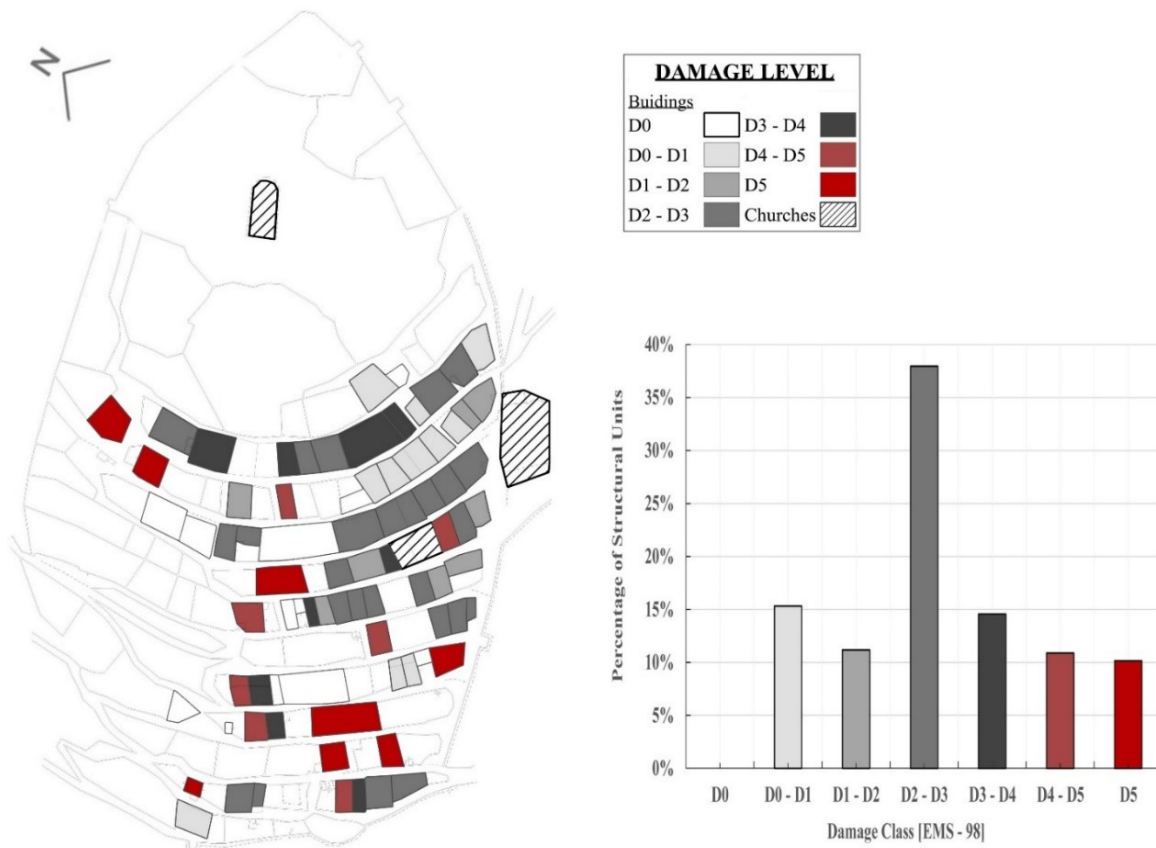


Figure 4.19: Mapping of the damage observed in Campi Alto di Norcia, according to the EMS – 98 scale.

Since non - structural elements are dangerous and can fall on the roads, causing deaths and injuries and blocking escape routes in case of evacuation (D’Orazio et al. 2014), (Bernardini et al. 2016), (Giuliani et al. 2019), Table 4.6 contains the “Non - Structural damage typologies” detected in Campi Alto.

Table 4.6: Percentage of “Non - structural damage typologies” detected, compared to the SUs analysed

Damage typology	Definition	Number of SUs detected	Number of SUs involved
External not dangerous	Detachment of plaster and false ceilings	67	100%
External medium dangerous	Falling of tiles and chimneys	60	89%
External heavy dangerous	Falling of balconies and cornices	23	34%
Internal	Furnishing elements	52	78%

Figure 4. 20 and Table 4.7 associates the distribution and the level of the damage with the position of the SUs in the masonry aggregates, to evaluate the aggregate effect in the seismic response of the buildings. Since the case study is formed by row aggregates, the SUs have an external (41%), internal (29%) or isolated position (30%). Among the latter, the damage level D4 – D5 and D5 is detected for the 18 and 41% of the buildings, while for other SUs the damage is equally distributed in the other levels (12%). The SUs with an internal position suffered mostly the damage level D2 – D3 (62%) and D1 – D2 (24%). On the contrary, the distribution of damage for external SUs is not regular and is split between D0 – D1 (23%), D2 – D3 (37%) and D3 – D4 (23%).

Table 4.7: Distribution of damage level in relation to the position of SU within masonry aggregates

Damage Level (DL) and general description		Isolated		Internal		External	
		SUs involved	[%]	SUs involved	[%]	SUs involved	[%]
D0	No damages	0	0%	0	0%	0	0%
D0 - D1	No structural damage and slight non-structural damage	2	12%	2	10%	7	23%
D1 - D2	No structural damage and roof tiles detachment	2	12%	5	24%	2	7%
D2 - D3	Slight structural damage and partial collapse of chimney and roof tiles' detachment	2	12%	13	62%	11	37%
D3 - D4	Large and extensive cracks in most walls and chimney fracture at the roofline	1	6%	1	5%	7	23%
D4 - D5	Partial structure failure of roof and floors	3.0	18%	0	0%	3	10%
D5	Total collapse of the building	7.0	41%	0	0%	0	0%
Total		17		21		30	

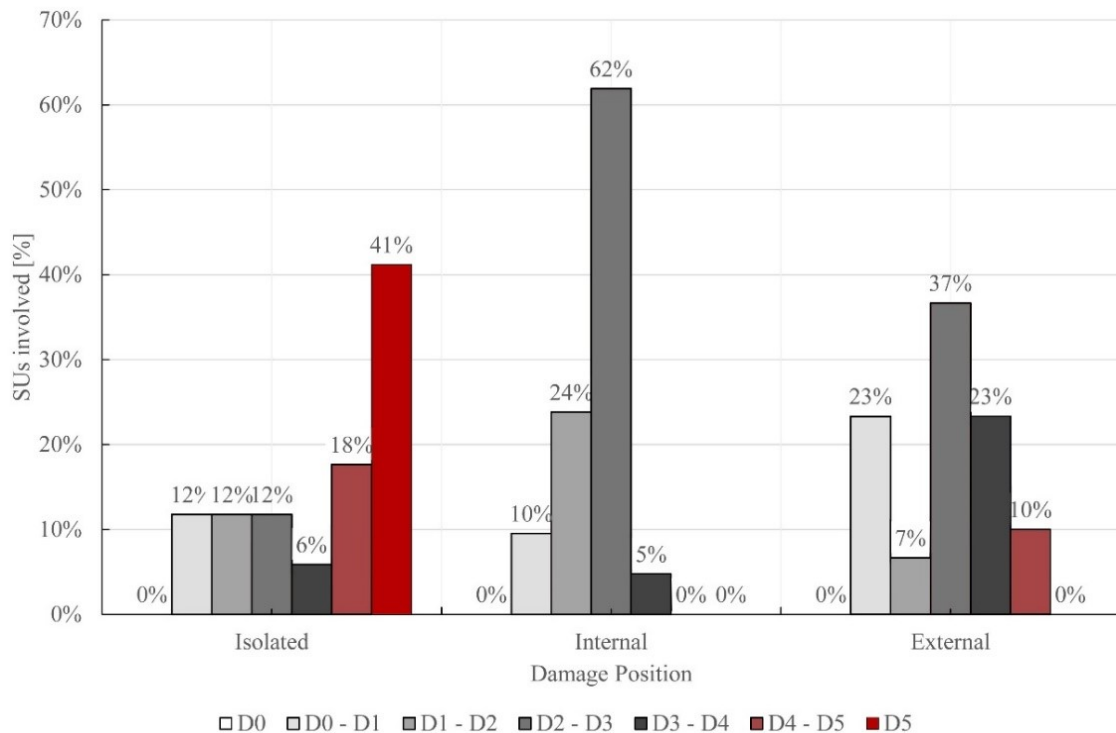


Figure 4. 20: Damage distribution detected according to the SU position within the masonry aggregate and the damage level.

The lateral constraints of the buildings, defined by the presence of other SUs, reduce their seismic vulnerability. In this sense, the structural and morphological evolution of the aggregate influence the degree of constrain, since external SUs with a high damage level show a bad connection system with the adjacent buildings (Figure 4.21).



Figure 4.21: Lack of connection between the SUs 142 and 143 (a), as well as among the SUs 151 and 152 (b).

The comparison between the retrofit interventions performed in the past with the damage detected allows to evaluate their efficiency, as analysed by Sbrogio et al. (2019) and Valluzzi et al. (2019). A clear example is the introduction of *reinforced concrete rings*, applied on the top floor, not connected to the underlying masonry (Figure 4.22) and the *connection systems* between masonry walls with rebars (Figure 4.23). This techniques, especially the use of reinforced concrete rings, which is currently not recommended because it increases the weight of the structure at a high altitude, if designed and executed correctly, can still provide a positive contribution to avoid local mechanisms. Figure 4.24 shows the damage caused by the lack of transverse connection in the double - leaf walls, comparing a retrofitted and a not retrofitted masonry wall.



Figure 4. 22: Inferior part of a reinforced concrete ring without elements (steel bars, glass fibers) that guarantee the connection with the masonry detected in the SU 155 (a); Portion of reinforced concrete ring, hanging from the roof of SU 183 (b).

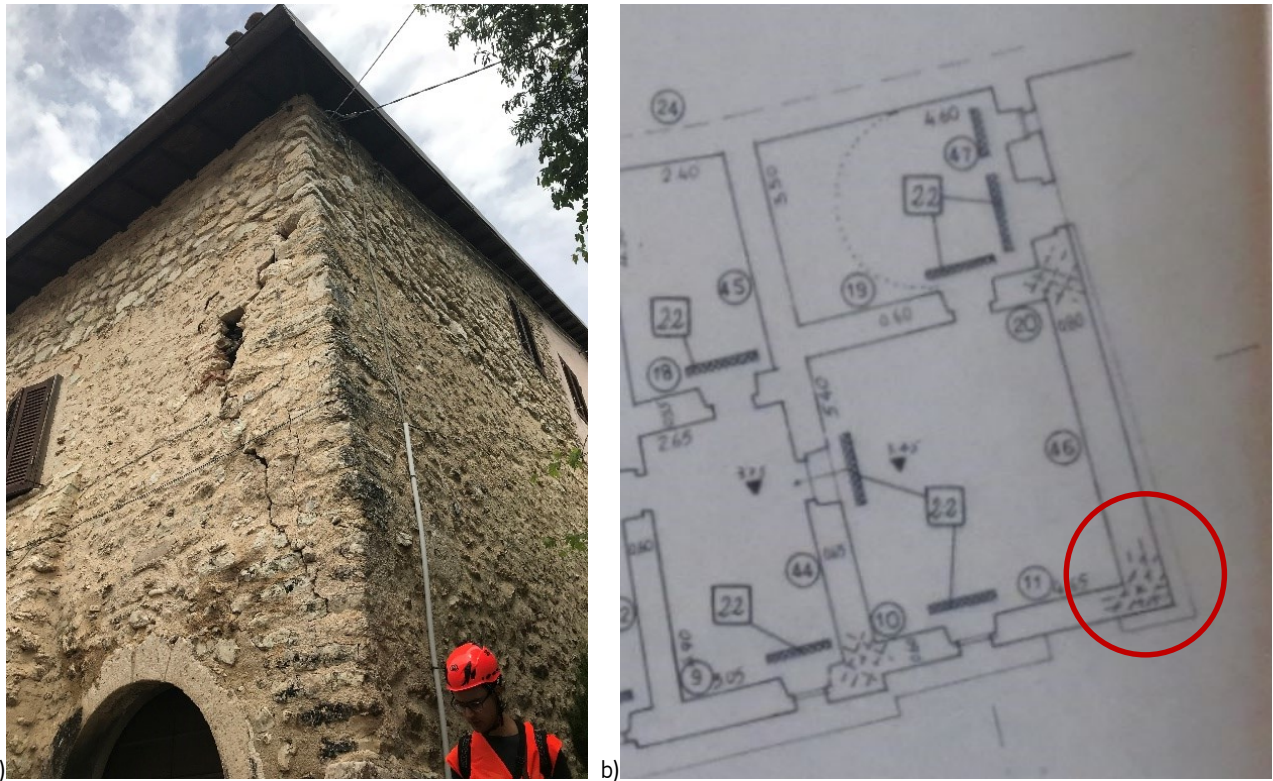


Figure 4.23: Partial detachment of the corner of the building SU 117 (a); Retrofitting Project of the first floor of the SU 117, which provides reinforced perforations to connect the two corner walls of the building. The damage detected shows that the strengthening was poorly performed or not performed (b).



Figure 4.24: Scheme of the failure mechanism due to masonry instability (a); Internal photo of SU 117.2, where the absence of transverse connections of the double leaves of the wall didn't allow the development of flexural/shear behaviour and the collapse occurred due to instability of the internal wall panel (b); The previous image is compared with the one collected in SU 168, in which an old retrofit technique, consisting of using the reinforced concrete with welded mesh and mortar injections, prevented an instability collapse.

Figure 4.25 shows a comparison performed, in a GIS environment, between the damage level (DL) of the SUs and the retrofit interventions carried out after the seismic events of 1979 and 1997. Most of the SUs, where the timber roofs and timber floors were replaced with concrete slabs ("storey stiffening" intervention marked in red),

show a Damage Level D4 – D5. The SUs reinforced with injections and steel stitching (“masonry reinforcement” marked in yellow) have an average DL around D3, while the SUs with the lowest DL are the ones characterized by the presence of ties on the façade (marked in green).

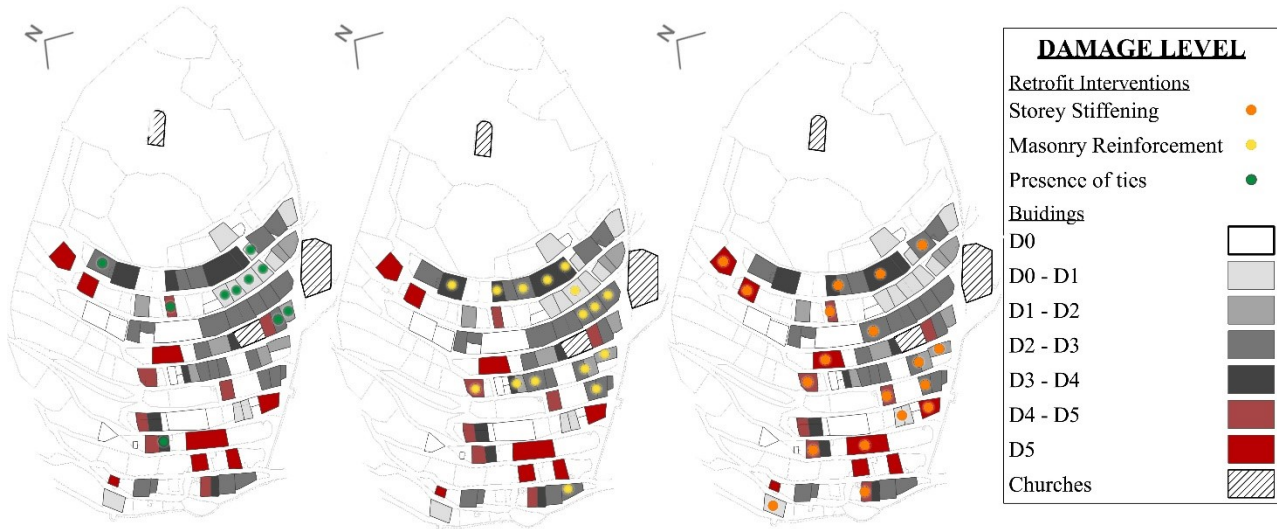


Figure 4.25: Comparison between the damage level and different retrofits typologies carried out before the seismic events of 2016.

4.5. Conclusions

The earthquake of 2016 and 2017 hit a large area of Central Italy, involving different regions and therefore many historical centres. Because of the geologic movement of the Apennines, these regions represent the most seismically active part of the country.

The case study of Campi Alto di Norcia was subjected to many destructive seismic events along its history, as testified by different documents collected, as well as by the construction system itself. The masonry aggregates are formed by small buildings, often reinforced with different retrofit techniques: i.e. buttresses built in the past and positioned at their ends or attached to the main facades, as well as stiffening of floors and roofs with concrete slabs, carried out after the earthquake late 1970s. Many of the modern strengthening proved to be unable to increase the seismic capacity of the buildings and even harmful.

The analyses of the earthquake characteristics, performed with accelerogram records, provide a further point of view on the seismic intensity that hit Campi Alto: the ground motion was higher than the value used by the Italian code for structural checks at the limit state of collapse. Moreover, the vertical component has influenced the buildings behaviour and the resulting damage detected, such as masonry failure. The seismic behaviour of masonry aggregate, evaluated theoretically in the previous chapters, is analysed in detail through the damage assessment. This structural system shows different damage typologies and on average their behaviour must be considered hybrid, influenced both by local mechanisms and in plane failures.

The position of the buildings within the aggregate is also an important feature to take into account in the assessment of seismic vulnerability, as the external and above all isolated buildings have suffered the greatest

damage. The historical analyses of the urban development of the historical centre can provide the degree of connection among the SUs, since it is hard to detect during the external survey of the buildings.

The connection of non- structural elements to the masonry structure is another important issue to consider for the seismic vulnerability assessment. The inspections have shown that even the least damaged buildings are subjected to detachment of chimneys and falling of broken tiles from the roofs. These elements can seriously injure, block escape routes and hinder the management of emergencies

The information collected in this phase are the base on which the improved vulnerability method and the seismic risk procedure are developed and calibrated.

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CHAPTER 5. Seismic vulnerability assessment of Campi Alto di Norcia

The four vulnerability methods described in chapter 2 are applied to the case study of Campi Alto di Norcia, which has been struck in the recent past by two relevant seismic events: the 24th August and 30th October 2016 earthquakes. The information obtained and shown in the previous chapters is used to perform the analyses. The reliability of the methods is assessed through confronting analytical results with post-seismic damage observations in masonry buildings.

Chapter outline

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5.1. Introduction

This chapter is directly related to the second and third objectives of the research: the four vulnerability methods selected and shown in Chapter 2 are applied to Campi Alto di Norcia (Italy) to determine the corresponding I_v and compare their accuracy in predicting damage scenarios. The purpose of this activity is to highlight possible limitations of selected methods, finally choosing the most accurate one for the considered case study. As discussed in Chapter 1, this is the first step of the second phase of the research activity, focused on the development of an improved vulnerability index methodology.

After carrying out a comprehensive survey of the building, the I_v evaluation of each SU of the historical centre is performed through a weighted sum of the vulnerability class of each parameter, according to Equation [2.3] and [2.4]:

$$I_v^* = \sum p_i \cdot c_i \quad \text{where} \begin{cases} c_i = \text{vulnerability class "i"} \\ p_i = \text{weight } i \end{cases} \quad [2.3]$$

$$I_v = I_v^* \text{ normalized to 100\%} \quad [2.4]$$

Given their good fit to the context of Campi Alto di Norcia the four approaches investigated herein are: the GDNT (1994), the Formisano et al. (2009), the Vicente (2010) and the Ferreira et al. (2012). Since these methods have been developed both for isolated buildings (GDNT II) and aggregates (Formisano et al., Ferreira et al., Vicente) their different assessments allow evaluating the aggregate effect, which is a fundamental requisite taking into account the morphological characteristics of Campi Alto di Norcia. The comparison of the different seismic vulnerability of the buildings with the observed damage provides an accurate representation of the mutual interaction among SUs and SA: the reliability of the parameters adopted to define the real structural performance of SUs in SA is, therefore, highlighted.

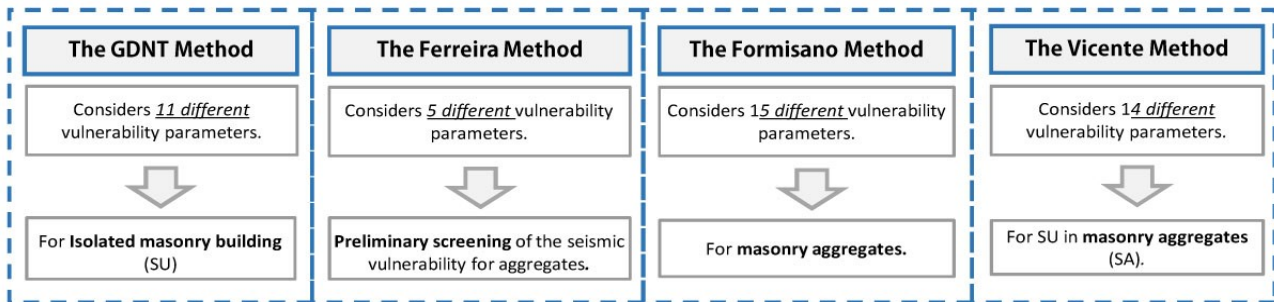


Figure 5.1: Vulnerability methods adopted for the case study of Campi Alto di Norcia.

The reliability of the selected statistical methods for the buildings' typologies of Campi Alto di Norcia is assessed by comparing the real damage data, recorded after the 2016 earthquake sequence, with theoretical damage scenarios evaluated through the above-presented I_v methods. The formulation proposed by (Bernardini et al. 2007), described before in Chapter 2, is adopted here to estimate the damages. The accuracy of the I_v method is assessed at the individual building level, measuring the relative error ($\overline{Err}_{mean,d}$) between the real and estimated damages, as well as through the theoretical damage evaluated with the seismic vulnerability of the whole historical city centre ($\overline{Err}_{mean,r}$). This analysis allows highlighting the difference between a classic direct vulnerability assessment and a further one, that considers the empirical vulnerability of a building obtained from a range between an upper and lower bound vulnerability value; The latter developed based on the mean vulnerability value of the construction typology analysed.

Figure 5.2 summarises the methodology adopted to assess the accuracy of the selected methods.

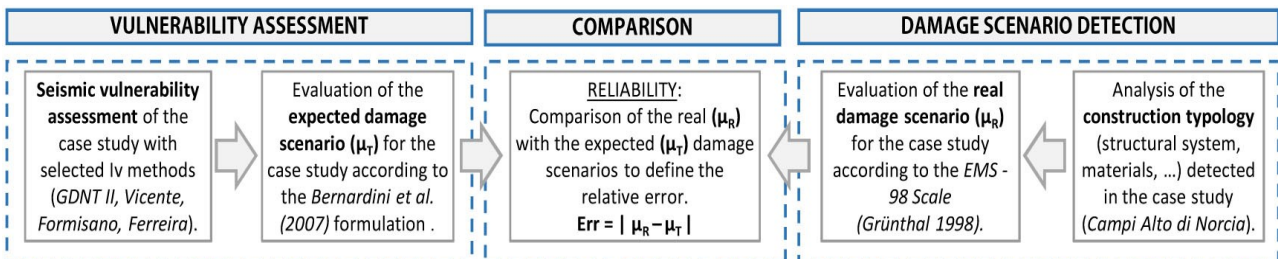


Figure 5.2: Framework followed to assess the accuracy of the four selected vulnerability index methods.

5.2. Application of the established vulnerability index (I_v) methodologies

5.2.1. The GDNT method

The GDNT (1994) method is composed of 11 vulnerability parameters (P1 to P11), which of each with four vulnerability classes (A to D, being A the best condition and D the worst one). The method was developed for isolated buildings, taking into account their geometry, structural and non-structural features, the floors and roof typologies, walls thickness and maintenance conditions. The maximum value of the achievable non-normalised I_v is 382.5.

Table 5.1 and Figure 5.3 present the distribution of the classes for the different parameters, from which it is possible to infer about the influence that every single parameter has in the overall assessment of the seismic vulnerability. As can be observed, the topographical conditions of Campi Alto di Norcia, assessed by Parameter 4, strongly penalises the vulnerability of the buildings, according to GDNT (1994) method. On the contrary, the small sizes of the buildings, assessed through Parameter 8, contributes to decrease the overall seismic vulnerability of the buildings.

Table 5.1: Relative frequency (expressed in %) of the number of structural units present in a class.

Score	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11
A	0.0%	0.0%	1.0%	0.0%	10%	28%	1%	100%	9.0%	10%	12%
B	27%	30%	7.0%	0.0%	70%	42%	13%	0.0%	70%	43%	67%
C	57%	60%	27%	0.0%	4.0%	22%	85%	0.0%	16%	37%	21%
D	16%	10%	64%	100%	15%	7.0%	0.0%	0.0%	4.0%	9.0%	0.0%

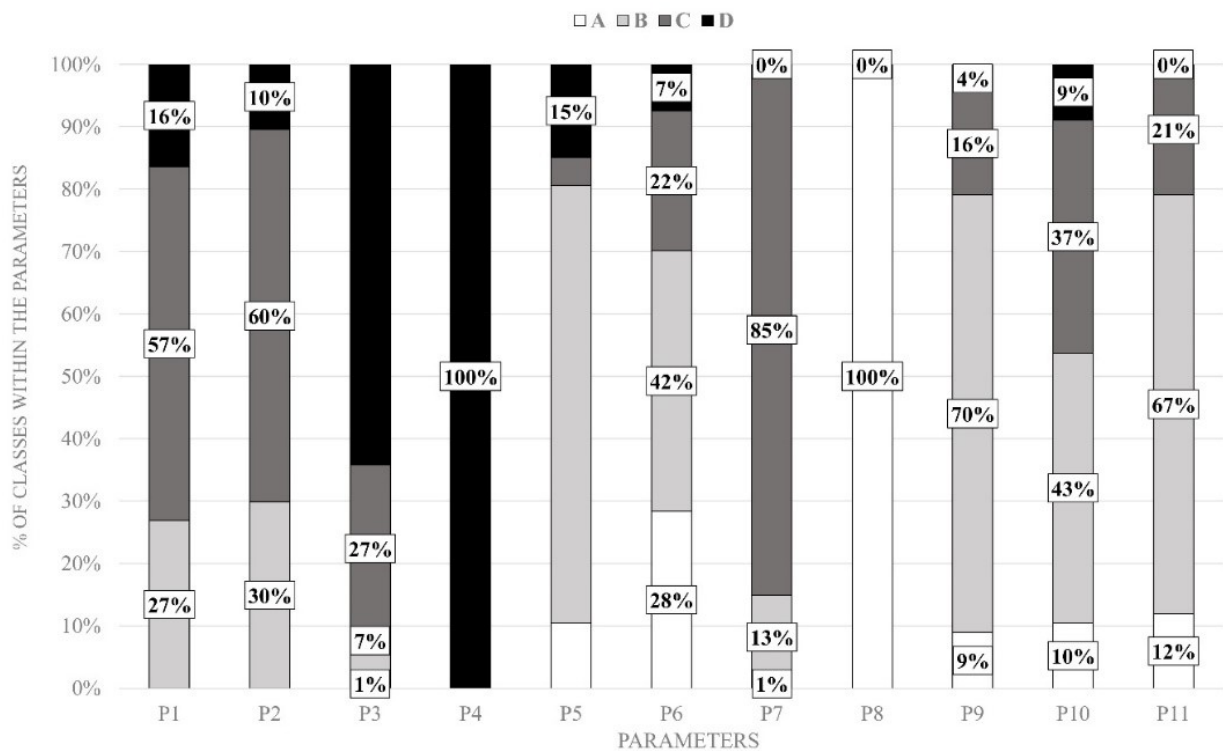


Figure 5.3: Vulnerability class distribution of each parameter using the GDNT method.

The application of the GNDT method to Campi Alto di Norcia resulted in an average I_v value of 47 and a Standard Deviation (SD) of 14. In terms of distribution, 6% of the I_v values – obtained resorting to Eqn. [2.4] – fall between 20 and 30 and 22% between 30 and 40. 43% of the buildings have a vulnerability index value between 40-50, 9% between 50-60, 7% between 60-70 and 12% between 70-80. The remaining 1% present a seismic vulnerability index value below 20. The statistical analysis of the vulnerability distribution obtained with the GNDT II method is shown in Table 5.2.

Table 5.2: Evaluation of the seismic vulnerability of 69 SUs present in Campi Alto di Norcia through GNDT II method

Statistical Analysis – GNDT II	
I_v average SU	47
I_v min SU	16
I_v max SU	79
Standard Deviation (SD)	14
I_v average + 2 SD	76
I_v average - 2 SD	18

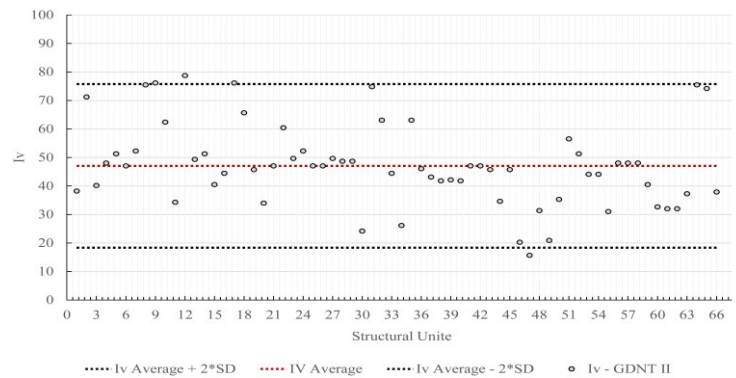


Figure 5.4 shows the distribution of the seismic vulnerability values over the study area, in terms of the position of SU and construction volume involved. In average, buildings with a volume of 186 m³ have a vulnerability index higher than 50, for a total amount of 3600 m³ of SUs involved, which corresponds to 29% of the entire building stock.

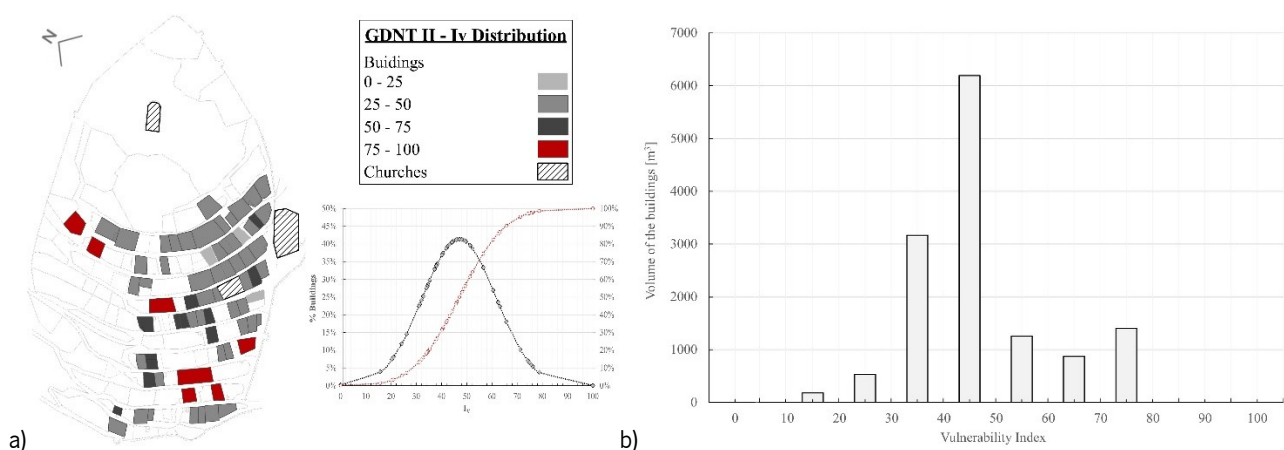


Figure 5.4: Distribution of the seismic Vulnerability Index (I_v) using the GNDT II method (a); I_v variation according to the volume of the buildings involved in the assessment (b).

5.2.2. The Formisano method

Intending to improve the (GDNT 1994) procedure by considering the SA effect and the interaction between SUs, Formisano et al. (2009) introduced five additional parameters: the position of the SU in the aggregate, the percentage of openings in the façade walls, the presence of staggered slabs, the structural difference between adjoining SUs and the interaction of neighbouring SUs with different heights. In Formisano et al. (2009), the maximum value that can be obtained for the non-normalised I_v is 515.25. Since the Formisano method is developed for SA, the isolated buildings are ignored in this analysis. For this reason, only the 50 SUs in aggregate position are considered in the following. As can be seen in Table 5.3, the distribution of the scores attributed to the vulnerability parameters is similar to the GDNT method until parameter 10. The interaction between buildings, which is evaluated by the Parameters 11-15. It was possible to observe that, in general, the position of the SU within the aggregate (Parameter 12) and the different structural typology of neighbouring SUs (Parameter 13) tend to lead to an increase on the vulnerability, whereas Parameters 11, 14 and 15 tend to contribute to reducing the vulnerability of the buildings. Table 5.3 and Figure 5.5 show the distribution of the score for the different parameters.

Table 5.3: Relative frequency (expressed in %) of the number of structural units for each vulnerability class.

Score	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14	P15
A	0%	0%	0%	13%	25%	1%	100%	9%	10%	12%	37%	0%	4%	31%	30%
B	25%	30%	0%	63%	46%	13%	0%	70%	40%	66%	48%	25%	4%	58%	63%
C	58%	60%	0%	6%	21%	85%	0%	16%	40%	22%	13%	0%	88%	7%	7%
D	16%	10%	100%	18%	7%	0%	0%	4%	9%	0%	1%	75%	3%	3%	0%

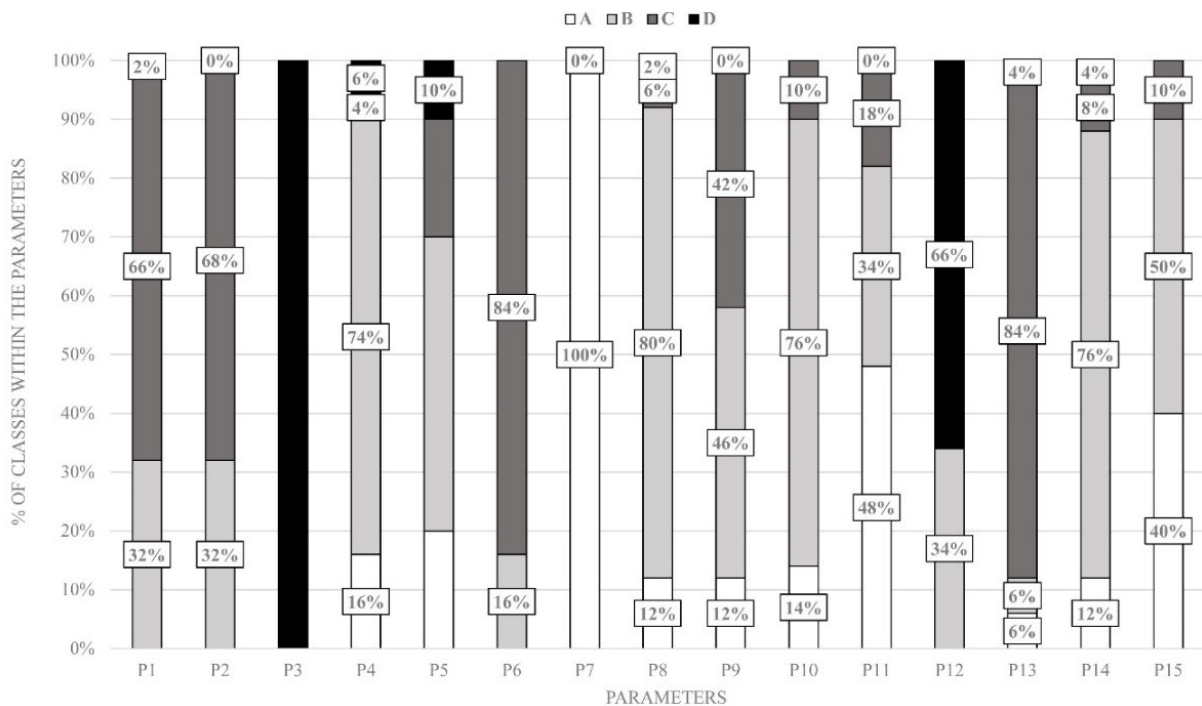


Figure 5.5: Vulnerability class distribution of each parameter using the FORMISANO method

In 13 % of the SUs assessed, the values of the I_v – computed according to Eqn. [2.4] – range between 0 and 10, in 44% of the cases, between 10 and 20, and in 35%, between 20 and 30. 2% of the buildings result in a vulnerability index equally distributed between 30-40 and between 40-50. As can be seen in Table 5.4, Formisano method leads to lower vulnerability values than the GNDT method: it was obtained an average I_v index of 17 and a Standard Deviation (SD) of 9.

Table 5.4: Evaluation of the seismic vulnerability of 50 SUs present in Campi Alto di Norcia through Formisano method

Statistical Analysis – Formisano	
I_v average SU	17
I_v min SU	0.0
I_v max SU	45
Standard Deviation (SD)	9.0
I_v average + 2 SD	34
I_v average - 2 SD	1.0

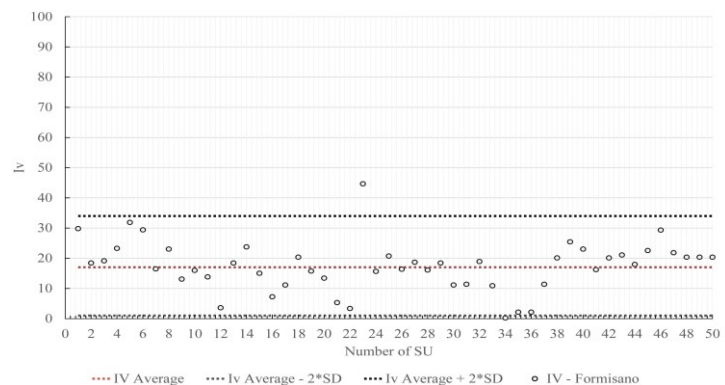


Figure 5.6 shows the distribution of the seismic vulnerability within the case study, according to the Formisano method. The vulnerability changes in a very low range and, once more, the highest values occur for the buildings located at the ends of the aggregates. On average, the aggregates with more volume are also the more vulnerable ones: 14 buildings with an average volume of 231 m³ present a vulnerability index higher than 20, for a total amount of 2300 m³ of SUs involved, which corresponds to 22% of the aggregates involved in the analysis (10612 m³).

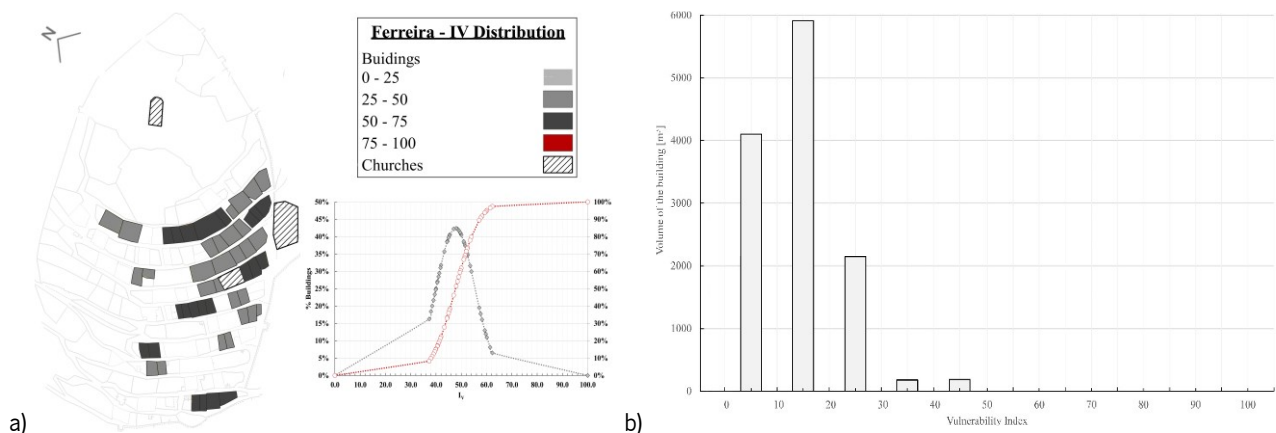


Figure 5.6: Distribution of the seismic Vulnerability Index (I_v) using the Formisano method (a); I_v variation according to the volume of the buildings involved in the assessment (b).

5.2.3. The Vicente method

Basing on the GDNT method, Vicente (2010) reorganised the evaluation parameter in four macro-classes related to the structural building system (P1 to P4), the irregularities and interaction (P7 to P10), floor slabs and roof (P11 to P12), and conservation status and other elements (P13 to P14). He also introduced two new parameters: Parameter 5 (P5) describes the number of floors, while parameter 10 (P10) defines the wall façade openings and alignments. The maximum I_v value according to this method is 650.

The Vicente form is applied to the entire historical centre. Therefore, 69 SUs are considered. The parameters that contribute the most to the increase of vulnerability values are the conventional strength (P3), the location of the building and type of foundation (P6), the aggregate position (P7) and the plan regularity (P8). On the contrary, the small distance between the walls (P4), the typology of floors and roofs detected, as well as the conservation condition of the SUs, tend to influence the vulnerability results positively. The distribution of the different scores within the parameters is given in Table 5.5 and Figure 5.7.

Table 5.5: Relative frequency (expressed in %) of the number of structural units per vulnerability class.

Score	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14
A	0%	0%	0%	100%	4%	0%	1%	12%	0%	25%	13%	16%	10%	12%
B	27%	30%	9%	0%	90%	0%	40%	19%	24%	30%	66%	73%	45%	72%
C	57%	58%	12%	0%	6%	0%	0%	30%	69%	31%	10%	10%	36%	16%
D	16%	12%	79%	0%	0%	100%	58%	39%	7%	13%	10%	0%	9%	0%

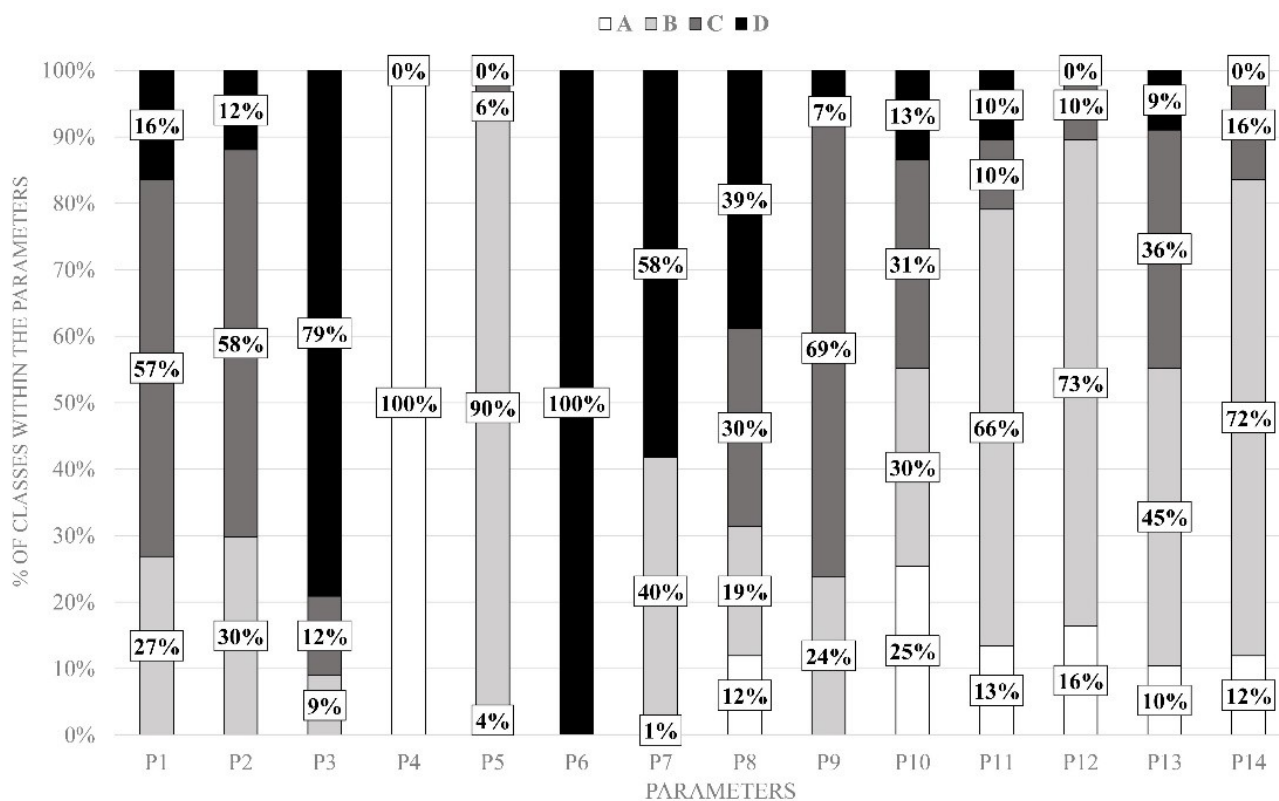


Figure 5.7: Vulnerability class distribution of each parameter using the VICENTE method.

The adoption of the Vicente method (2010) resulted in an average vulnerability index value of 41 and a Standard Deviation (SD) of 11. In terms of distribution, 4% of the evaluated buildings shows an I_v index between 10 and 20, 12% between 20 and 30, 30% between 30 and 40, and 35% between 40 and 50 (Table 5.6). The remaining the buildings, about 19%, present an I_v higher than 50. Again, for the sake of completeness, the average I_v value obtained when considering only the 50 structural units (SUs) belong to the structural aggregates (SAs) evaluated with the Formisano method is 39, with a standard deviation of 9.

Table 5.6: Evaluation of the seismic vulnerability of the 69 SUs present in Campi Alto di Norcia using Vicente method

Statistical Analysis – Vicente	
I_v average SU	41
I_v min SU	17
I_v max SU	63
Standard Deviation (SD)	11
I_v average + 2 SD	22
I_v average - 2 SD	19

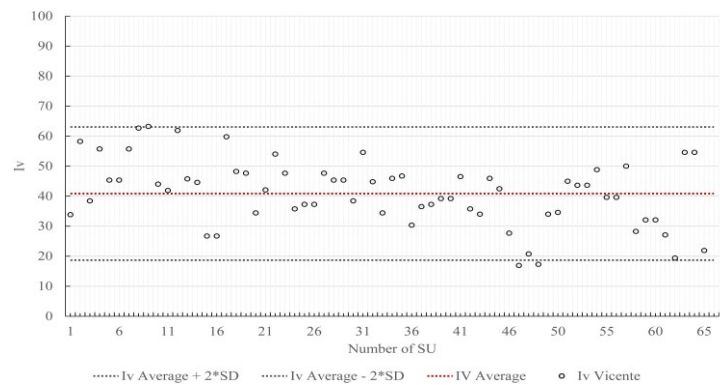
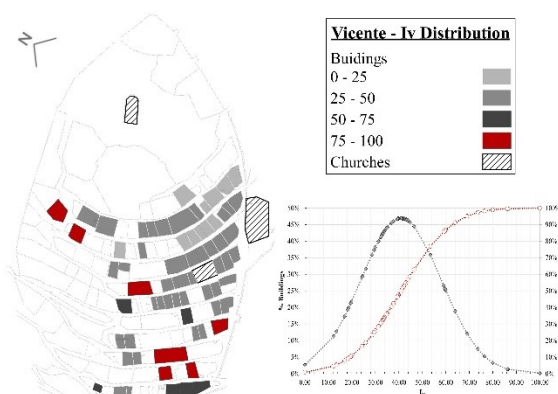
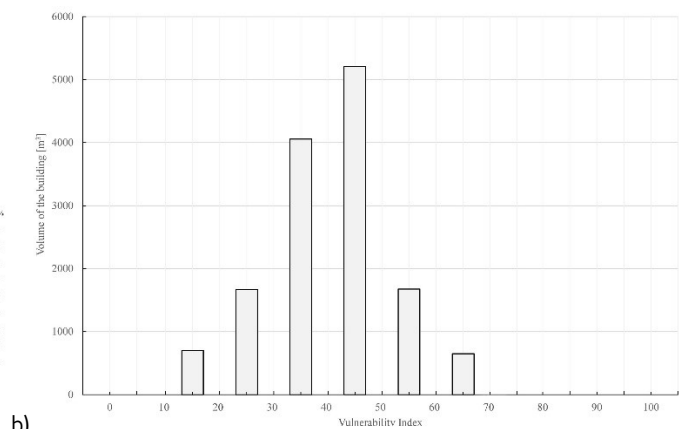


Figure 5.8 shows the distribution of the seismic vulnerability values obtained with Vicente method. Eleven buildings with a total volume of 7600m³ present a vulnerability index higher than 50, which corresponds to 60% of the total volume, about 12400 m³. 6%, 15% and 32% of the entire built volume is associated with I_v values in the range between 20 -30, 30-40 and 40-50, respectively, see Figure 5.8 (b).



a)



b)

Figure 5.8: Distribution of the seismic Vulnerability Index (I_v) using the Vicente method (a); I_v variation according to the volume of the buildings involved in the assessment (b).

5.2.4. The Ferreira method

The Ferreira et al. (2012) method resorts to five semi-qualitative parameters (i.e. the quality of masonry, the presence of misalignments among openings, the presence of irregularities in elevation, the organisation of building plan and the location and soil category) to estimate the seismic vulnerability of aggregates. The value of the I_v according to this approach range from 0 to 212.5. 16 masonry aggregates are evaluated in this work using Ferreira method. As given in Table 5.7 and Figure 5.9, Parameters P4 and P5 (the plan regularity and the location and soil quality, respectively) are those that contribute the most to raise the global seismic vulnerability of the aggregates assessed herein. Oppositely, the parameters concerning the misalignment of openings (P3) and the regularity in height (P3) present a little influence on the global vulnerability of the assessed aggregates.

Table 5.7: Relative frequency (expressed in %) of the number of structural units in each class.

Score	P1	P2	P3	P4	P5
A	0.0%	44%	38%	19%	0.0%
B	34%	41%	53%	31%	0.0%
C	47%	13%	9.0%	6.0%	0.0%
D	19%	3.0%	0.0%	44%	100%

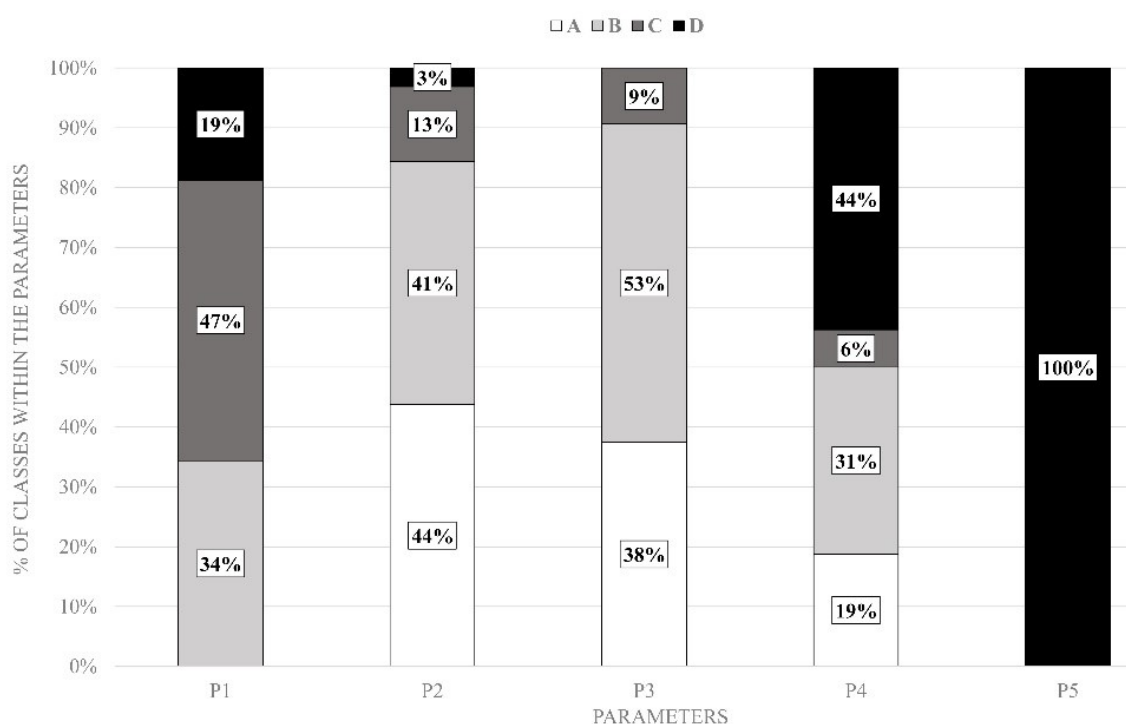


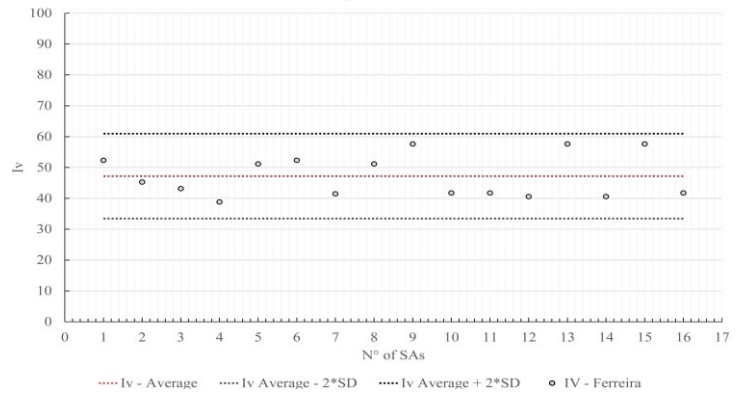
Figure 5.9: Vulnerability class distribution of each parameter using the FERREIRA method

The assessment of the historical city centre of Campi Alto di Norcia using the Ferreira method resulted in the following distribution: 6% of the SAs present a value within the range 30-40, 50% between 40-50, and 44%

between 50-60. As a general remark, aggregates an average vulnerability value of 47 and a Standard Deviation (SD) of 7.

Table 5.8: Seismic vulnerability results of the 16 SAs in Campi Alto di Norcia assessed using Ferreira method

Statistical Analysis – Ferreira	
I_v average SU	47
$I_{v,min}$ SU	39
$I_{v,max}$ SU	58
Standard Deviation (SD)	7.0
I_v IV average + 2 *SD	61
I_v average - 2 *SD	33



According to the quick screening of the seismic vulnerability performed with the Ferreira method, the total aggregate volume is evenly distributed between the vulnerability index ranges of 40 to 50 and 50 to 60, see Figure 5.10.

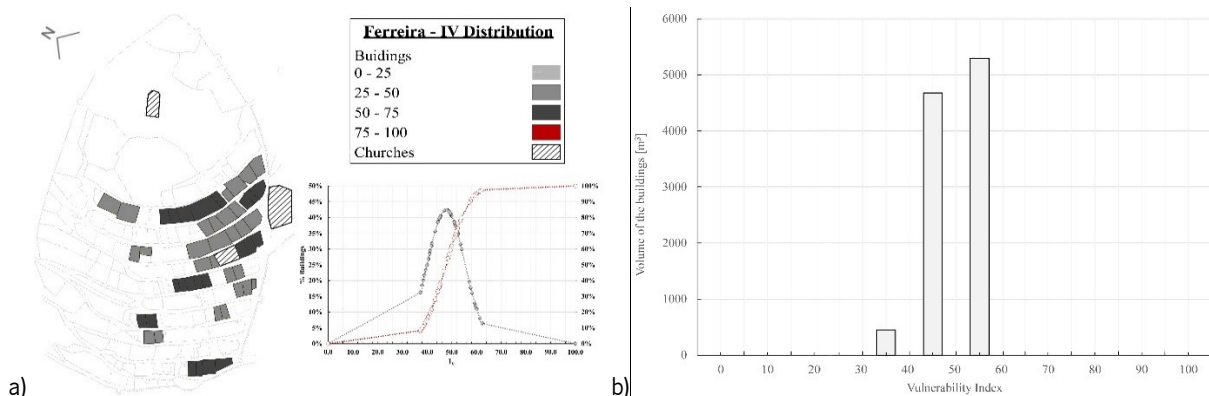


Figure 5.10: Distribution of the seismic Vulnerability Index (I_v) using the Vicente method (a); I_v variation according to the volume of the buildings involved in the assessment (b).

To summarise the above-discussed results, from the four methods analysed the GDNT method is the one that leads to the highest seismic vulnerability values of I_v . Regarding the trend of the data, Ferreira approach is the one that presents a lower scattering of the results. Focusing specifically on the two methods that take into account the aggregate effect, Formisano method produces very low vulnerability values, while Ferreira and Vicente's methods show, on average, moderate to high values. The results demonstrate the influence that the weights and scores of the different methods own in the vulnerability assessment outputs. I.e., Formisano's method tends to produce lower vulnerability values because P4 (i.e. the position of the building within the masonry aggregate) and P1 (i.e. the percentage of opening areas) tend to impact positively the seismic performance of the buildings assessed. Similarly, P7 of the Vicente method (i.e. aggregate position and

interaction) reduces the seismic performance of the SUs positioned at the corners, improving otherwise the behaviour of SUs inside the aggregates.

Figure 5.11 shows the distribution of the I_v obtained with the different methods analysed in this work. In a general comment to the result shown in Figure 5.11 it is worth highlighting the relevant differences found between the vulnerability index values obtained with different methods, which, as said, have been developed and calibrated based on different premises.

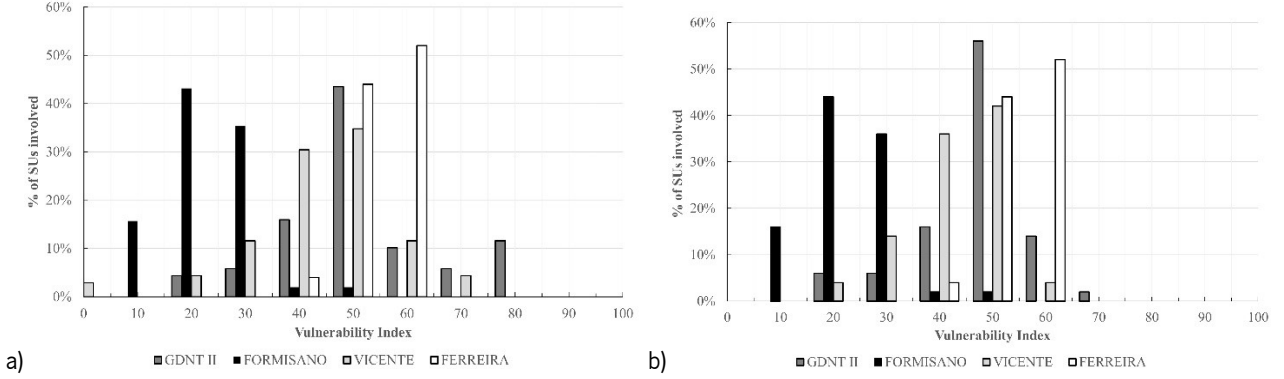


Figure 5.11: Distribution of the I_v for 67 SUs (a); Distribution of the I_v obtained for the 50 SUs in the aggregate (b).

5.3. Analysis of the accuracy of the seismic vulnerability methods

The accuracy of the seismic vulnerability assessment analysed in this work to the buildings' typologies of Campi Alto di Norcia is assessed by comparing the observed damage data recorded after the 2016 earthquake sequence, quantified according to the EMS-98 Scale (Grünthal 1998), with the mean damage grades obtained using the above-presented I_v methods. For such, the formulation proposed by (Bernardini et al. 2007) is adopted here:

$$\mu_D = 2.5 + 3 \cdot \tanh\left(\frac{I + 6.25 \cdot V - 12.7}{Q}\right) \cdot f(V, I), \quad \text{for } 0 \leq \mu_T \leq 5 \quad [2.1]$$

$$f(V, I) = \begin{cases} e^{\frac{V}{2}(I-7)} & I \leq 7 \\ 1 & I > 7 \end{cases} \quad [2.2]$$

Basing on post-seismic damage evaluations of irregular stone masonry buildings and on (Sandi and Floricel 1995), a ductility factor of $Q = 2.5$ is adopted for masonry buildings with sufficient ductile behaviour. The parameter V is evaluated through the expression proposed by Vicente et al. (2011):

$$V = 0.56 + 0.0064 \cdot I_v \quad [5.1]$$

Given that the intensity map of the October 2016 seismic events occurred in Campi Alto di Norcia region has been produced using the MCS scale (Musson and Cčić 2011), it is necessary to convert this intensity – equal to 8.5 (Galli et al. 2017) – to the European Macroseismic (EMS-98) scale, through Eq. [5.2] (Margottini C., 1992):

$$I_{EMS-98} = 0.734 + 0.814 \cdot I_{MCS} \cong 8.00 \quad [5.2]$$

The intensity obtained from Eqn. [5.2] is then used to input the subsequent analysis procedure. Thus, the accuracy of each I_v method is assessed through the mean relative error of the values achieved for the whole building stock analysed. The uncertainties affecting the methods are both epistemic and aleatory, being related to the characteristics of each aggregate, the evaluation of vulnerability classes or the construction typologies (Spence et al. 2003).

In an attempt to overcome some these issues, particularly the uncertainty associated with the evaluation of the vulnerability parameters that compose the methods, the vulnerability of each building is evaluated not as a single value but as a range of values defined based on the average characteristics of the study area. Following this procedure, upper and lower bounds of the I_v are defined for each SU, considering the SD of the vulnerability assessment of Campi Alto di Norcia. Resorting to Equation [2.3], it is then possible to achieve a reliable and credible range for the expected damage.

The accuracy of each statistical method is subsequently evaluated as the minimum relative error obtained from comparing the observed and the analytical damage, which is defined considering the lower and the upper I_v values. A range of the initial level of expected damage is then established to perform the parametric analysis of the seismic vulnerability of the considered construction typology, evaluating the influence of the I_v methods in the seismic response of the buildings. The accuracy of the I_v method is assessed, therefore, not only at the individual building level (a direct measure of $\overline{Err}_{mean,d}$) but at the global level of the whole historical city centre (measure performed using the full sample range $\overline{Err}_{mean,r}$), as given by Equation [5.3].

$$\begin{aligned} \overline{Err}_{mean,d} &= \left(\frac{\sum Err}{n} \right) \\ \overline{Err}_{mean,r} &= \left(\frac{\sum \min(Err_-; Err_+)}{n} \right) \end{aligned} \quad [5.3]$$

Where Err is the error for a given building, defined as the difference between analytical and real damage detected in the case study in absolute value, n the number of buildings, and Err_- and Err_+ the errors in a given building as the minimum difference between analytical damage, considering the upper and lower bounds, and the real damage. The maximum and minimum error is evaluated by increasing and reducing I_v in a magnitude that is given by the variance found in the overall vulnerability assessment results. This way, it is possible to address and mitigate the uncertainties associated with the vulnerability assessment process.

Figure 5. 12 summarises the procedure adopted in this work to evaluate the accuracy of the analysed vulnerability index methods to Campi Alto di Norcia. This procedure is applied to each of the three I_v methods adopted, comparing the results obtained and assessing the lowest relative error (Table 5.9). As can be seen in

Figure 5.13 none of the methods was able to estimate the observed damage of Campi Alto di Norcia with high accuracy, resulting in differences higher than 0.85 in average, for all procedures. Considering a probable theoretical damage range, the relative errors decreased, remaining otherwise higher than 0.58. As already said, results highlight the low accuracy of the methods if used under very different conditions of those assumed in their calibration processes.

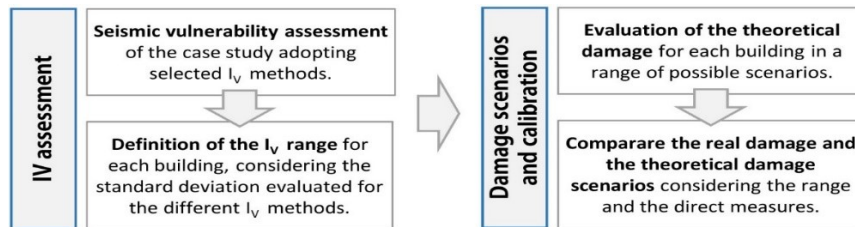


Figure 5. 12: Procedure adopted to evaluate the reliability of vulnerability assessment methods.

Table 5.9 – Evaluation of the I. method reliability in the case study analysed

Iv Methods	I. Methods accuracy (direct measure)		I. Methods accuracy (assuming a range)	
	Average relative errors	Variance relative errors	Average relative errors	Variance relative errors
GDNT II	0.87	0.67	0.67	0.37
Formisano	1.13	0.72	0.72	0.55
Vicente	0.85	0.61	0.58	0.34
Ferreira	0.92	0.78	0.72	0.35

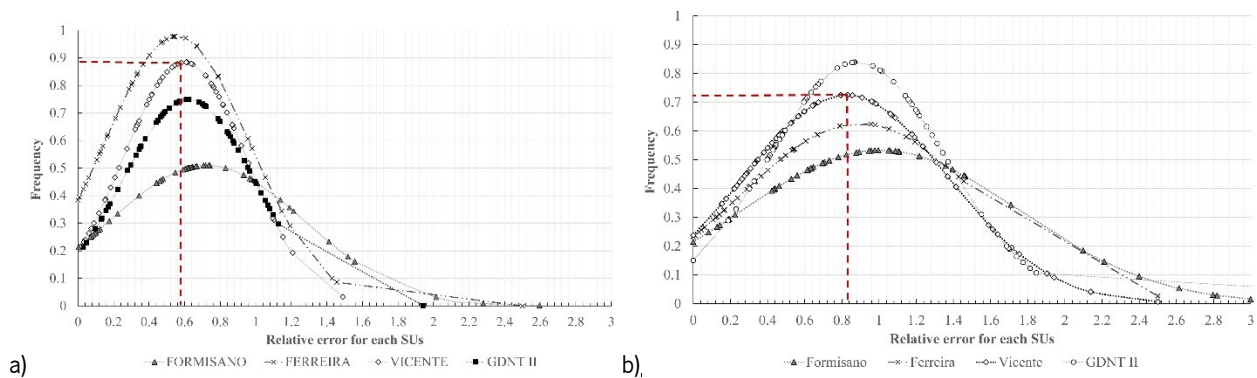


Figure 5. 13: Relative error distribution of the IV methods obtained from the direct measure (a) and the full sample range (b). The dashed red line indicates the average error obtained with Vicente's method.

In order to check whether the relative error is caused by an *overestimation* or *underestimation* of the real damage, it is necessary to analyse with more detail the individual damage distributions obtained with each one of the four approaches. Considering the mean value of the I_v range for each building in the SA, it is possible to compare results of the different methods at single building and urban scale levels. Figure 5.14 shows the comparison between the analytical and the real damage distribution in Campi Alto di Norcia, carried out at building scale by evaluating, for each SU, the distance between the real and analytical damage value. As can

be observed in Figure 5.14, while the values of the real damage are quite scattered, the distribution of the estimated damage is concentrated around the average values: this point out the limitations of adopting Equation [2.3] to estimate high or low damage classes for different vulnerability values.

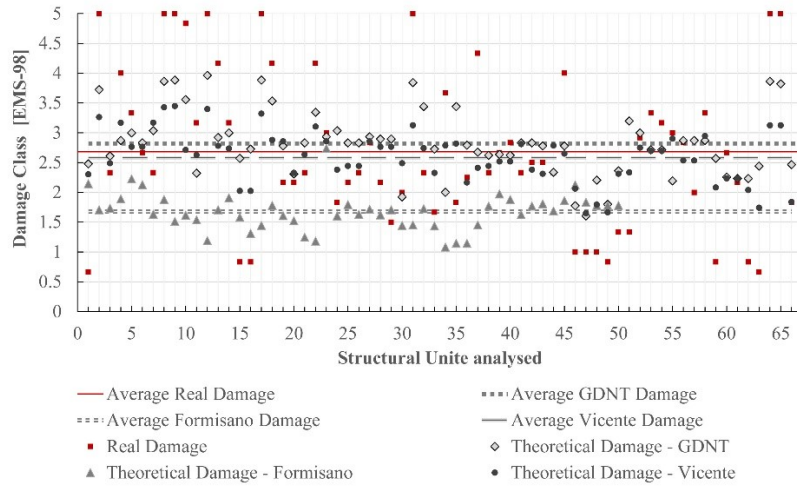


Figure 5.14: Comparison between the Real Damage detected in the case study and the Theoretical Damage evaluated through different vulnerability methodologies for each SUs.

Figure 5. 15 illustrates the comparison between real and analytical damage for Campi Alto di Norcia using the four above discussed vulnerability assessment methods. Some preliminary conclusions can be drawn: the Formisano et al. (2009) method provides less conservative vulnerability results, presenting thus a propensity to underestimate the damage; the average value of the observed damage lies in the middle between the GDNT (1994) and the Vicente (2010) methods, presenting the latter, however, the advantage of being able to catch the influence of the aggregate effect. It is important to stress that these results are related to a single intensity value. Extensions are therefore needed in order to get a more general overview of the actual seismic vulnerability of the case study. In any case, by considering the average vulnerability values obtained with the different approaches, it is possible to produce different damage scenarios for the study area.

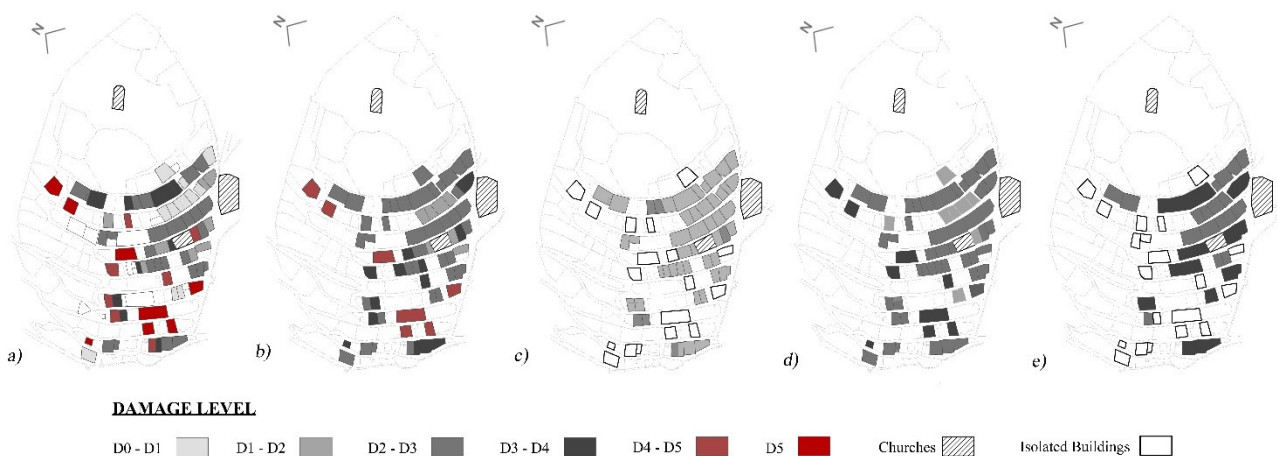


Figure 5. 15: Comparison between the real damage detected in the case study (a) and the theoretical damage scenarios evaluated through the GDNT (b), Formisano (c), Vicente (d) and Ferreira method (e) for each SU.

The vulnerability curves presented in Figure 5.16 show how these methods overestimate or underestimate the real damage even for different macroseismic intensities. The curve representing the real damage of the case study for different seismic intensity is a simple forecast, achieved through the average vulnerability value obtained from the real damage obtained through Equation [2.1].

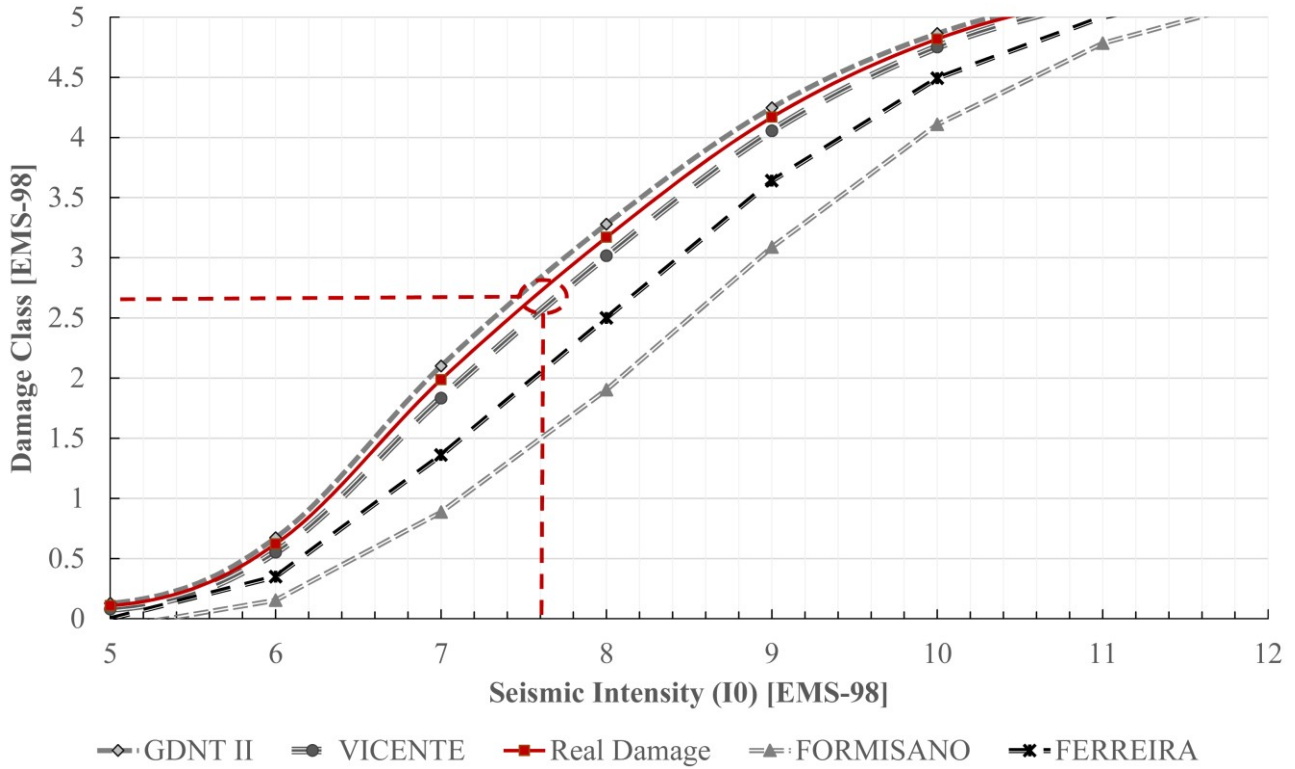


Figure 5.16: Vulnerability curves of the case study for different seismic intensities using the IV methods selected; the damage and the intensity of the seismic events of October 2016 is highlighted in red.

5.4. Conclusions

Seismic vulnerability assessment methods based on statistical evaluations and damage observations are suitable tools for urban scale analysis. Exploiting limited information and resources, they provide a first screening on the fragility of the area for a given seismic scenario.

From the analysis of the results presented in this chapter, it is possible to draw some conclusions about the main limitations of the methods analysed in this thesis, and, as a whole, of the simplified index-based vulnerability assessment formulations. Firstly, their high dependence on the building characteristics and on the seismic scenario(s) originally used to calibrate and validate them. Another limitation is related to the definition of a single value for the vulnerability, which can potentially increase the difference between the real and expected/ analytical damage. A range of vulnerability values, rather than a discrete value, can be a reasonable way to account for uncertainties in the assessment, related to buildings' typologies, structural and architectural features and, besides, to the specific expertise and perception of the technicians performing the evaluation. In this sense, as discussed in Section 5.3, upper and lower bound damage scenarios, related respectively to a maximum and minimum analytical damage value, have been adopted here.

The estimated damage distribution is concentrated around the middle classes of the EMS-98 scale, not allowing to consider low (D0) or high (D5) damage classes, even for very low or high vulnerabilities. This leads to an increase of the relative error between the analytical and real damage, resulting from a misestimation of Bernardini's formulation (Bernardini et al. 2007) for low seismic intensities and not from a limitation of the vulnerability estimation itself.

Regarding the performance of the four methodologies analysed herein, the GDNT form was the one that led to the most conservative scenarios in terms of damage. In the opposite direction, the Formisano and Ferreira methods have revealed a tendency underestimate the vulnerability and, in consequence, the damage. From the four approaches, the Vicente method proves to be the most accurate one for the building typology studied in this work and, therefore, it will be taken as the basis for the development of the improved method, in Chapter 6.

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Chapter 6. Proposal for an improved seismic vulnerability assessment method

Basing on the damage detection of the case study and the vulnerability assessments performed, an Improved vulnerability index method is developed and proposed in this chapter. It includes new parameters for the out of plane behaviour of the buildings in aggregate, as well as a new procedure for evaluating the quality of the data collected on-site. The methodology adopted for the classes and weight definition is described and discussed in detail herein.

Chapter outline

6.1 Introduction

6.2 Proposed seismic vulnerability assessment method

6.3 Definition of the new seismic vulnerability classes

6.3.1 The Development of the façade benchmark

6.3.2 The Evaluation of the capacity curves

6.3.3 The Definition of the vulnerability classes

6.4 Definition of seismic vulnerability parameters weight

6.4.1 Application of the Improved method in the case study

6.4.2 The calibration procedure

6.4.3 The expert opinion

6.4.4 Comparison of the results

6.5 The Information Quality Index (IQ)

6.6 Conclusions

6.7 References

6.1. Introduction

The results achieved in the previous chapter shown that of the four vulnerability assessment approaches considered in this thesis, the Vicente (2010) method is the one that, for construction typology of the case study, leads to most accurate results. Since the damage observed in Campi Alto often results of combined in-plane and out-of-plane failure mechanisms, the proposed method must take into account these two types of failure (i.e., out-of-plane and in-plane) in order to capture and accurately assess the vulnerability of Campi Alto's buildings. Based on the exposed, the Vicente (2010) method is reorganised and extended in this chapter, introducing new parameters for the out of plane behaviour of structural units in building aggregates.

An extensive parametric numerical evaluation, aiming at the assessment and quantification of the influence of the new vulnerability parameters, is performed. Firstly, a façade benchmark is defined, and the rocking

mechanism of several façade configurations are analysed using a kinematic approach (Casapulla et al. 2017). Then, capacity curves are developed for each parameter and subsequently used to obtain vulnerability classes. Moreover, the weights of the new parameters are evaluated to assess their importance in the seismic vulnerability of the building. This is carried out in two steps: (i) the value of the weights suggested by other vulnerability index methodologies for similar parameters are used as kick-off values; (ii) the weights are then calibrated by comparing the '*observed*' damages caused by the 2016 earthquake with the '*analytical*' ones obtained with the Bernardini et al. (2007) formulation. Statistical analysis is, therefore, performed within the context of Bayesian inference (Carlin and Louis 1997).

To overcome the limits of the calibration procedure and to verify the consistency of the statistical analysis, the values of the new weights are compared with those obtained by expert opinion using the Analytical Hierarchy Process (AHP) proposed by (Saaty 1980). This procedure allows quantifying the relative importance of the parameters based on the subjective opinion of experts. Thus, the values of the weights derived mathematically based on observed damage data are compared with the value suggested by experts with extensive experience in this field.

Finally, an additional IQ index is introduced in the calibrated procedure accounting for uncertainties related to the lack of information due to limited access or inspections. The application of different I_v methods in the case study of Campi Alto, performed in the previous chapter, highlights the advantages and disadvantages of the Empirical/Expert base methods: their reliability progressively decreases when applied to construction typologies that are substantially different of those for which the methods have been developed and calibrated. Moreover, the uncertainties related to the lack of information due to limited access during the inspections and the experience of the technician carrying out the survey strongly influence the vulnerability assessment of the masonry building. This issue is testified by the difference between the relative error of the direct ($\overline{Err}_{mean,d}$) and range ($\overline{Err}_{mean,r}$) measure – i.e. by taking into account the uncertainties, the variance among real and analytical damage scenario decrease, if the seismic vulnerability of the building is assessed within the range of probable values. The knowledge uncertainty is, therefore, quantified through a parameter that measures its influence in the final vulnerability assessment, and an iterative analysis involving the change of the uncertainty range of the parameters based on the observed damages is adopted.

To guarantee an accurate application of the improved method, a guideline on how to correctly evaluate the vulnerability parameters was prepared and provided in *Annex C*.

Following the organisation of the thesis, much of the information described in the previous chapters are used in this chapter for the development and calibration of the improved method. Figure 6.1 summarises the procedure followed for this development.

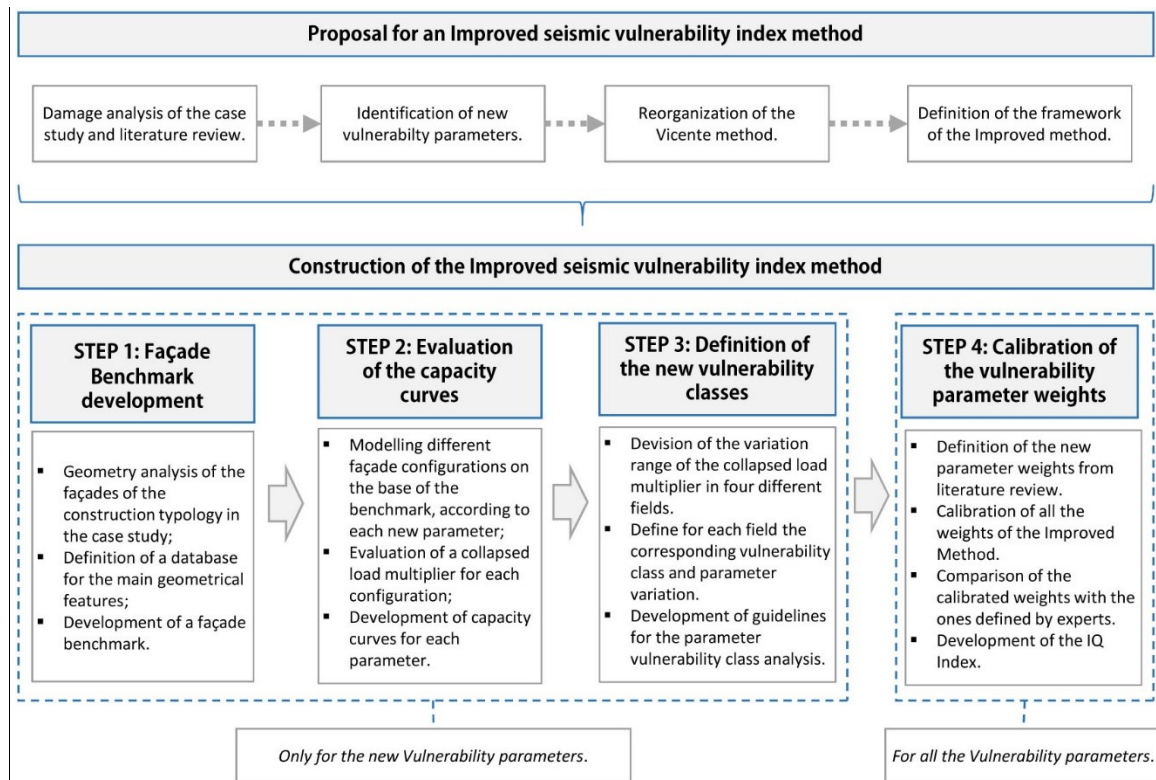


Figure 6.1: Procedure followed for the definition of the improved method

6.2. Proposed seismic vulnerability assessment method

The damage typologies, identified in the case study of Campi Alto di Norcia and described in detail in Chapter 4, show the importance of taking into account the activation of local mechanisms in the vulnerability assessment of masonry buildings in aggregate. Moreover, in Chapter 5, the Vicente (2010) method appears to be the most reliable vulnerability method for the construction typology analysed. This method takes into account the out of plane behaviour of the building by addressing the structural features that guarantee the so-called box behaviour of the SUs through parameters P1 “Type of resisting system”, P4 “Maximum distance between walls”, P10 “Wall facade openings and alignments”, and P12 “Roof type”.

The objective of the improved method is to reorganise the framework of the Vicente (2010) approach and to extend it with new vulnerability parameters specifically thought to consider the most common local mechanisms among those observed in the damage dataset used in this thesis.

It is worth noting that, by definition, the parameters that compose any statistical/empirical-based seismic vulnerability assessment method must be, as much as possible, unequivocal and easy to check on site. In this sense, the new parameters address only structural feature that can be identified through an external survey of the masonry building.

The new parameters have been selected based on the work developed by many authors in different vulnerability index methods – namely (Lagomarsino and Podestà 2004), (Ferreira et al. 2017), (Rapone et al. 2018) –, and considering the structural performance of the construction typology hit by the seismic events of Central Italy (2016).

Table 6.1 shows the four final parameters, defined after several preliminary analyses.

Table 6.1: Description of the new vulnerability parameters

Vulnerability Parameter		Short Description
P1	Façade Slenderness	Ratio between the height (H) of the façade and its thickness (b) at the base ($FS = H/b$).
P2	Façade Openings	Ratio between the openings' area and the total area of the wall, measured at each floor.
P3	Façade Connections	Presence of metal ties at each floor.
P4	Corner Detachment	Type of building's corner, considering the presence of ties on the two corners' sides, the masonry quality and the regularity of the cornerstones.

Past earthquakes have shown that the activation of local mechanisms in masonry aggregates is governed by the failure of individual SUs, which is typically caused by the lack of connection between the different structural elements resulting of the growth process of the aggregate (Lagomarsino and Magenes 2009). Therefore, the selected vulnerability parameters are developed for single structural units and don't take into account local mechanisms involving parts of different buildings in the same aggregate. This kind of failure is also too particular and complex to be analysed with an index-based vulnerability assessment methodology. In the same way, the parameter P4 "Corner Detachment" should only be applied when there is a clear geometrical corner of the masonry structure, such in the cases of the corner walls of the external structural units of a masonry aggregate or of isolated buildings. The Improved Method is defined by 15 parameters arranged into four different (Table 6.2).

Table 6.2: Summary Table of the vulnerability parameters adopted by the Improved Method.

Vulnerability Parameters		Short description
GROUP 1: Global Behaviour		
P1	Quality of the resisting system	Quality of the vertical resisting system and wall connections
P2	Conventional strength	Walls' shear strength assuming box behaviour
P3	Number of floors	Number of positive floors
P4	Horizontal diaphragms	Quality of floor type considering stiffness and connection with the walls
P5	Roof typology	Weight and roof typology (thrust)
P6	Location of building and type of foundation	Topographical conditions and quality of the foundation soil
GROUP 2: Local Behaviour		
P7	Façade Slenderness	Ratio between the height (H) of the façade and its thickness (b) at the base ($FS = H/b$)
P8	Façade Openings	Ratio between the openings' area and the total area of the wall, measured at each floor
P9	Façade Connections	Presence of metal ties at each floor
P10	Corner Detachment	Type of building's corner, considering the presence of ties on the two corners' sides, the masonry quality and the regularity of the cornerstones
GROUP 3: Aggregate effect		
P11	Aggregate position and interaction	Position of the SU in the SA
P12	Plan configuration	Length/ width ratio of the building plan
P13	Height regularity	Mass variation in elevation and the presence of arcades or towers
GROUP 4: Conservation Status and other elements		
P14	Fragilities and conservation state	Masonry quality and cracking scenario
P15	Non-structural elements	Type and characteristics of the non-structural elements

Improved Method (2020)

6.3. Definition of the new seismic vulnerability classes

6.3.1. The Development of the façade benchmark

A façade benchmark is defined for the study of the out of plane behaviour of the construction typology of the case study. This geometrical model will base the development of different façade configurations, according to the vulnerability parameter analysed. The façade benchmark is evaluated considering the geometrical features of the entire building stock of Campi Alto. As described in Chapter 3, the settlements in these territories are built along the slopes of hills, influencing the structure of the buildings. They are usually developed on three different levels, perpendicular to the slope of the hill: the ground floor – partially underground – and two additional upper floors (Figure 6.2).



Figure 6.2: View of Campi Alto di Norcia, a medieval town located in the Apennines of Central Italy before the earthquake of 2016 (a); Image of the construction typology of Campi Alto di Norcia (b).

From the geometrical point of view, the façade of the downstream road is characterised by relatively big entrances (2.50 x 1.50m) and relatively small openings (1.20 x 1.00m), while the wall's thickness change over the height, with average values around 120 cm at the ground floor (without the buttress) and reductions up to 70 cm in the upper levels. The openings disposition is not regular and mostly misaligned both horizontally and vertically. The lateral façades follow the shape of the hill and masonry buttress can be identified, increasing the average thickness of the wall at the first storeys. The façade of the upstream road has in average wall thickness of 70 cm, and small windows (0.60 x 0.60m) and doors (1.80x1.00).

A database of the main geometrical features of the façades of the entire historical centre is created by measuring the dimensions of the piers and the spandrels at each floor of the buildings. Based on the average values of these geometrical features, a façade benchmark is developed and presented in the following (Table 6.3).

Table 6.3: Façade geometry benchmark

Façade Geometry Benchmark							
Floor	Thickness [m]	Height [m]	Spandrel height		Spandrel width		
			First spandrel [m]	Second spandrel [m]	First spandrel without the openings [m]	Halfway spandrel without the openings [m]	Second spandrel without the openings [m]
1	1.20	3.10	1.10	2.30	4.50	1.80	7.00
2	0.90	2.80	1.10	2.30	5.00	1.80	7.00
3	0.70	2.80	1.10	2.30	5.00	1.80	7.00

Since the dimensions and the position of the openings are not regular, a percentage of the opening is considered for each floor of the façade benchmark, changing the initial size of the piers walls and spandrels. The position of the openings, shown in Figure 6.3, is therefore only indicative, aiming at showing the reference height in which they are positioned.

The configuration of the façade described in Table 6.3 is used to evaluate the parameters “*Façade slenderness*”, “*Façade Connection*” and “*Facade Openings*”. Besides, by connecting two façades orthogonally, the benchmark used for the analysis of the “*Corner Detachment*” parameter is created. According to the construction typology analysed, the lateral façade is usually deprived of openings due to the presence of the abutments of the barrel vault. For this reason, the corner benchmark doesn’t have openings on one side at the first level.

Figure 6.4 shows the 3D rendering of the façade benchmarks. The model variations for each parameter is described in the next paragraph.



Figure 6.3: Construction typology of the case study (a); Geometry of the Façade benchmark (b).

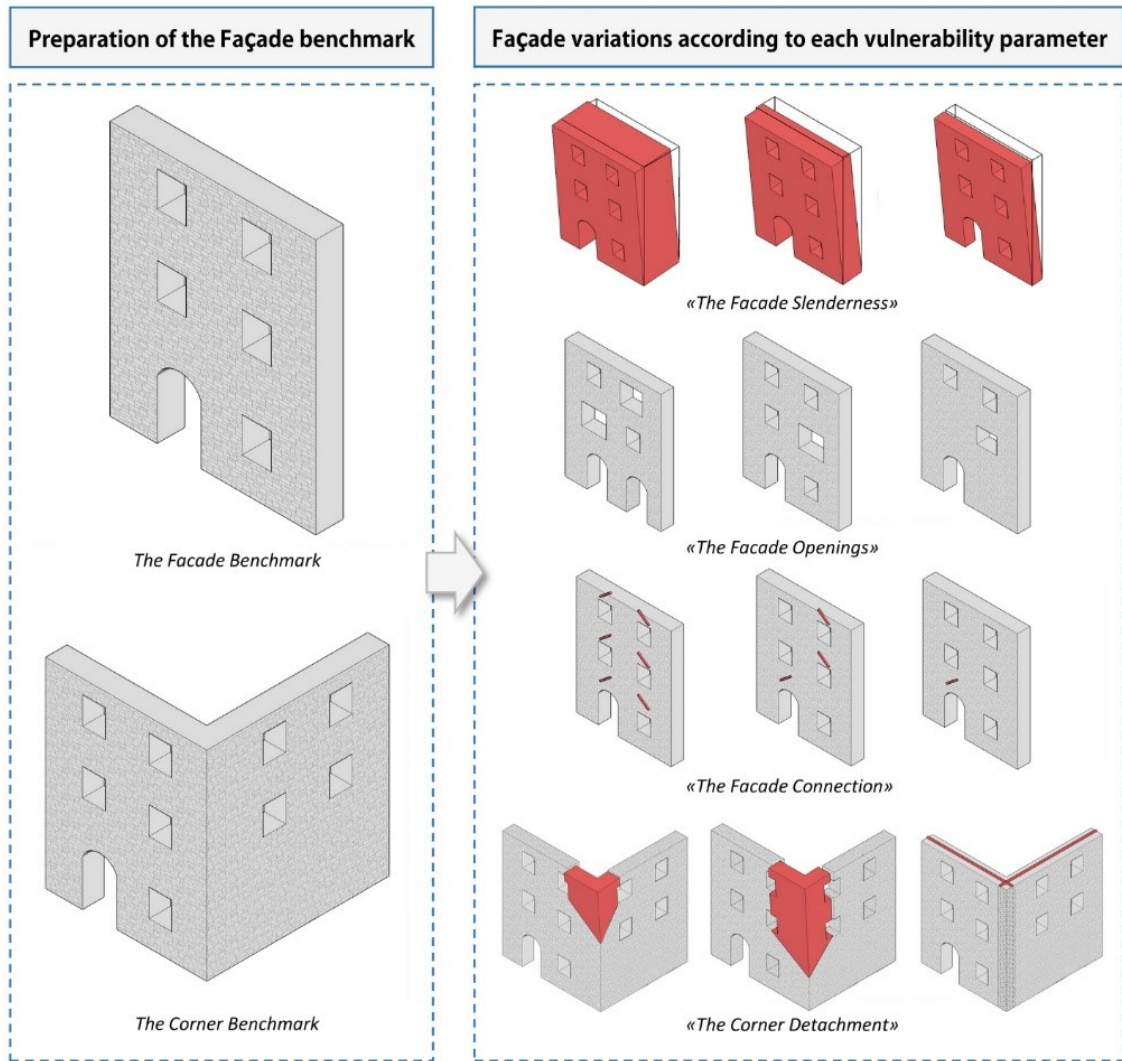


Figure 6.4: Procedure for the façade benchmark evaluation and model variations for each parameter.

6.3.2. The Evaluation of the capacity curves

Starting from the façade benchmark, capacity curves are developed for each vulnerability parameter. For such, a series of limit analysis (Giuffrè 1992) considering the rocking mechanism of different façade configurations has been performed. These configurations are defined based on the geometrical and structural features that influence the new vulnerability parameters, according to the damage observed onsite (refer to Chapter 4). For each configuration, a collapsed load multiplier is assessed to define the value of the rocking activation of the façade. The results are then arranged from the smallest to the highest value, developing a capacity curve.

According to the kinematic approach, the collapse multiplier (α_0) of the horizontal loads is defined, assumed proportional to dead and live loads, which induces rocking activation for the macroelement (Figure 6.5a). In case of complex multiple-block mechanisms, the value of α_0 can be estimated by means of the Virtual Work, by matching the work of external forces (L_{fe}) with that exerted by internal forces (L_{fi}), such as frictions between the blocks or metal tie stretch:

$$L_{fe} = \alpha_0 \left(\sum_{i=1}^n P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j} \right) - \sum_{i=1}^n P_i \delta_{y,i} - \sum_{h=1}^0 F_h \delta_h = L_{fi} \quad [6.1]$$

$$\alpha_0 = \frac{\sum_{i=1}^n P_i \delta_{y,i} - \sum_{h=1}^0 F_h \delta_h + L_{fi}}{\sum_{i=1}^n P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j}} \quad [6.2]$$

Being P_i the weight of i -th block that compose the wall, P_j the j -th generic load applied on the façade and F_h is a generic external force. While $\delta_{x,i}$, $\delta_{y,i}$, $\delta_{x,j}$ and δ_h are the virtual displacements of different forces P_i , P_j and F_h .

The collapse multiplier is, therefore, dimensionless and depends only on the geometry of the system and not on the mechanical properties of the macroelement's materials. However, since the rigid rotation around a point involves infinite compression stresses, the masonry resistance is taken into account through the displacement of the rotation point with respect to the external corner, according to the compressive stress reached.

This procedure is taken by the Italian Code (NTC 2018) (C.M. n.7 2019) for the seismic safety assessment of existing masonry structures and is based on several theoretical and experimental studies (De Felice and Giannini 2001), (D'Ayala and Speranza 2003), (Lagomarsino 2015), (Sorrentino et al. 2017). Moreover, several local mechanisms affecting the SUs that compose the masonry aggregates – based on the damage observed in historical centers after earthquakes (Milano et al. 2006) – are evaluated through the kinematic-based approach. Such a procedure allows understanding the structural features and boundary conditions that mostly influence the out of plane behaviour of masonry buildings, providing a robust base to the analyses. The evaluation of possible local mechanisms for masonry buildings solve, furthermore, the issue related to the identification of possible kinematic configurations.

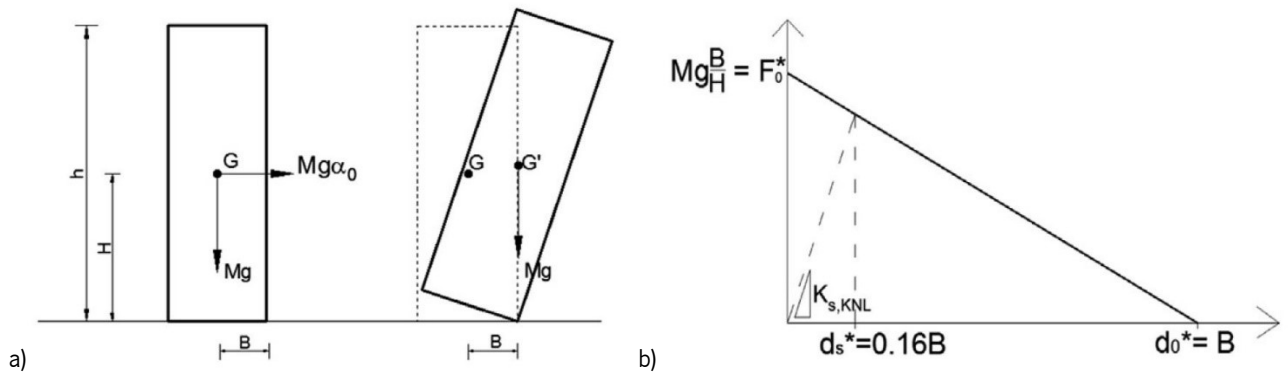


Figure 6.5: Rocking activation and overturning condition (a); Capacity curve - Kinematic Non-linear approach (b) (Berto et al. 2018).

The Capacity curves, developed for each vulnerability parameter analysed, consider two failure typologies: the overturning mechanism for the “*The Façade slenderness*”, “*The Façade Connection*” and “*The Façade Openings*” and the corner overturning mechanism for the “*The Corner Detachment*”. The formulations proposed

by Milano et al. (2006) are taken as reference to show the loads considered for the overturning mechanism [6.3] and corner detachment [6.4]:

$$\alpha_0 = \frac{\sum_{i=1}^n W_i \frac{s_i}{2} + \sum_{i=1}^n F_{vi} d_{vi} + \sum_{i=1}^n P_{si} d_i + \sum_{i=1}^n T_i h_i - \sum_{i=1}^n F_{Hi} h_{vi} - P_H h_i}{\sum_{i=1}^n F_{vi} h_{vi} + \sum_{i=1}^n W_i y_{Gi} + \sum_{i=1}^n P_{si} h_i} \quad [6.3]$$

Where W_i represents the weight of the i -th rigid block, F_{Hi} and F_{vi} the horizontal and vertical component of the thrust forces of the i -th floor, P_{si} the load of the i -th floor, T_i metal tie of the i -th floor and P_H the horizontal force of the roof. Parameters s_i , d_{vi} , d_i , h_i , h_{vi} , h_i and y_{Gi} are the lever arms of the considered forces.

$$\alpha_0 = \frac{W x_G + F_v d_v + P d_p + P_{v1} d_1 + P_{v2} d_2 + (T'_1 + T'_2) h - F'_H h_v - (P_H + P'_{H1} + P'_{H2}) h}{W y_G + F_v h_v + (P + P_{v1} + P_{v2}) h} \quad [6.4]$$

Being W the weight of the rigid wedge, F_H and F_v the horizontal and vertical component of the thrust forces, P_H and P_v the horizontal and vertical component of the i -th load at the top of the macroelement, T_i i -th metal tie and P_H the horizontal force of the roof. Parameters d_v , d_i , h_i , h_v , h , y_G and x_G are the lever arms of the considered forces.

The weight of the blocks and the forces acting on the façade benchmark are defined based on the information reported in Chapter 3. It is worth noting that the most vulnerable configuration was identified beforehand by analysing the orientation of the slabs Figure 6.6 shows the scheme of the two local mechanisms considered for the capacity curve evaluation.

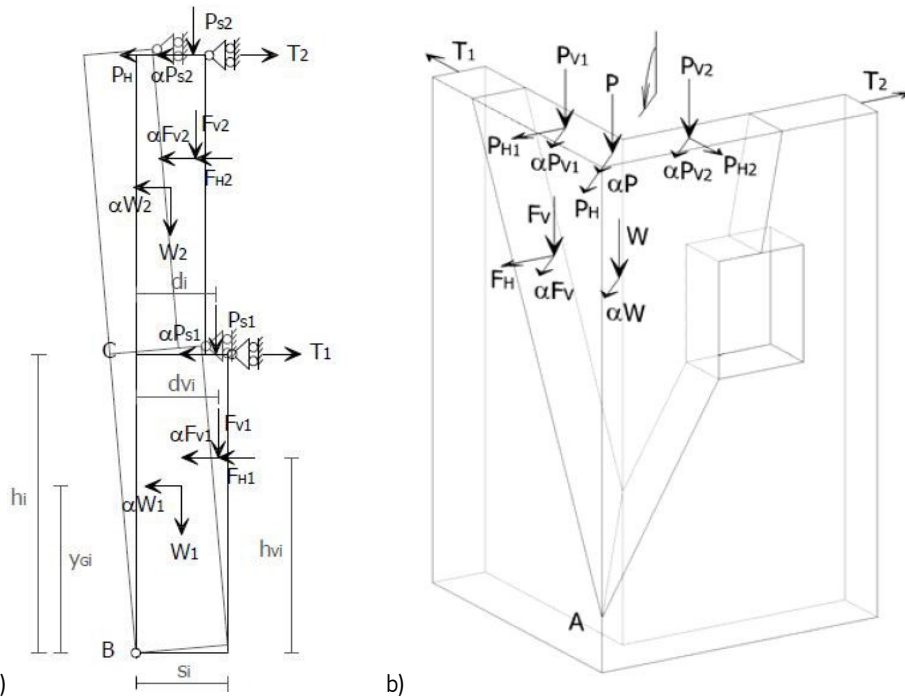


Figure 6.6: Scheme of the overturning mechanism (a) and corner detachment (b) according to (Milano et al. 2006).

The Façade Slenderness

Since the slenderness is defined as the ratio between the height (H) of the façade and its thickness (b) at the base ($FS = H/b$), the analysis is carried out by changing the thickness of the façade benchmark. In this sense, several façade configurations are evaluated, keeping all the other variables unchanged and without taking into account the presence of metal ties – their contribution is considered in the parameter “*Façade Connection*”. For each configuration, the collapse load multiplier (α_0) that induces the rocking activation of the macro element is computed. Finally, the capacity of the system is described by a hyperbolic curve with a descending trend, where the value of the α_0 obtained for each configuration is in the y-axis, and the corresponding slenderness value is given in the x-axis. In this sense, the collapse load multiplier reaches high values for wall slenderness equal to 5, while slenderness higher than 16 leads to the failure of the facade due to loss of balance (Figure 6.7). These values are comparable with the slenderness ranges suggested walls by different authors (Rondelet 1830), (Breyman 1885), (Giuffrè 2010) for the stability of masonry, as well as by the modern construction codes for existing masonry buildings (EN 1996-1-1 2005), (C.M. n.7 2019). The same analyses are performed considering the retraction of the overturning point (hinge) due to the exceeding the compressive strength of the masonry. Other collapse effects, such as the failure of the masonry due to lack of quality of the material is taken into account in the parameter’s vulnerability classes. Finally, the presence of a scale effect in rigid body motion, which makes the larger of two blocks with same slenderness ratio (H/b) more stable than the smaller one, is taken into account (Berto et al. 2018). It should be noticed that the presence of buttress attached to façade increases its base thickness, providing a low slenderness value. In this sense, the overturning mechanism can involve the portion of the facade above the buttress, rather than the entire façade. The scale effect of the façade is taken into account by evaluating the “Complete overturning” option, which allows to increase the score of the parameter with the same slenderness of the façade and therefore the final vulnerability, as reported in *Annex C*. Figure 6.8 shows the activation of the partial overturning of the façade, a collapse mechanism that has been actually detected in the case study. The capacity of the system is evaluated through the analysis of the movement of the overturning point above the final portion of the buttress, considered on average at the base of the first floor for each façade configuration.

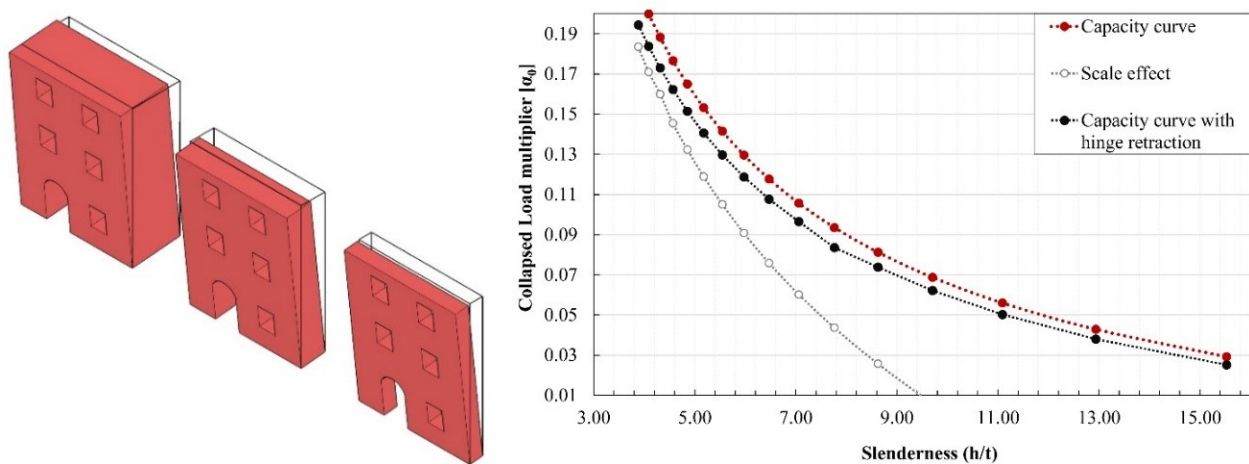


Figure 6.7: Capacity curve of the vulnerability parameter “Façade Slenderness”.

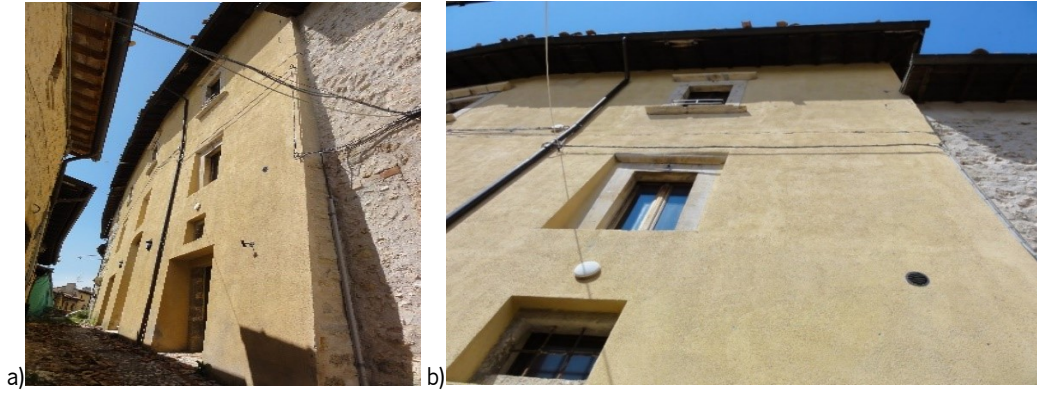


Figure 6.8: Activation of the partial overturning of the façade, triggered by the different façade thickness and less connection of the wall with the horizontal elements, a mechanism that is characterised by a horizontal crack in the final part of the buttress (SA23_SU190).

The Façade Openings

The façade openings influence the distribution of horizontal actions within SUs in aggregate: their dimension suggests the strength of the perimeter walls of the buildings, while their misalignment among the SUs indicates the presence of staggered floors. Vulnerability methods, such as Formisano et al. (2009) and (Vicente 2010), take into account the distribution of the openings in relation to the global behaviour of the buildings. In this sense, following the damage detected in the case study, these structural characteristics are analysed in relation to the single building to evaluate their influence on the rocking mechanism. The vulnerability parameter related to the “*Façade Openings*” is examined considering 40 different configurations in which different percentage of openings (p_i) are applied on each floor: the position of the openings is not taken into account, and three percentage classes, $p_i < 25\%$, $25\% < p_i < 50\%$ and $50\% < p_i < 75\%$, are identified. In the first configuration (C1), the openings percentage of 75% is adopted for the first floor and 25% for the other ones, while in the configuration (C40) 25% of openings are considered for the first and second floor and 75% for the other ones. The analyses show that the collapse load multiplier is reduced by a high percentage of openings at the ground floor level, regardless of the percentage of openings on the other levels (Figure 6.9). The presence of several openings on the ground floor moves the centre of gravity of the rigid body upwards, thus diminishing the collapsed load multiplier. In this sense, the retraction of the overturning point (hinge) due to the exceeding the compressive strength of the masonry reduces the capacity of the system.

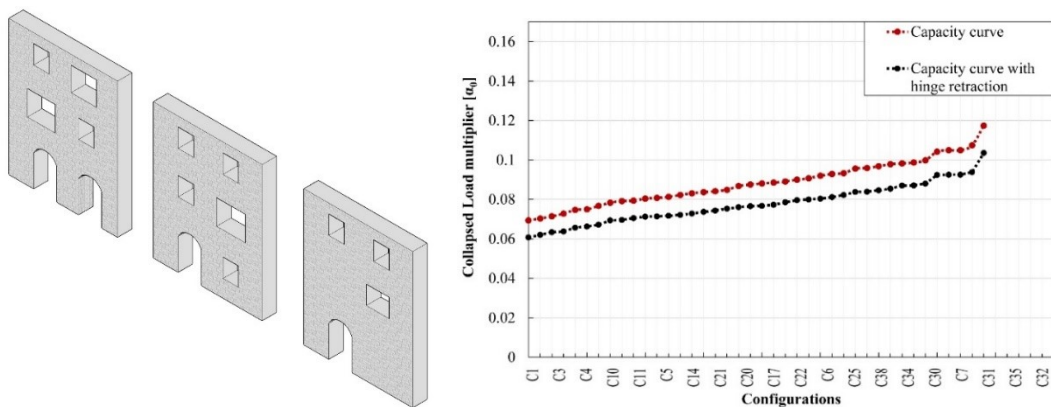


Figure 6.9: Capacity curve of the vulnerability parameter “Facade Openings”.

Façade Connections

To analyse the façade connections, the distribution of the metal tie-rods is taken into account. These elements play a significant role in ensuring the “Box Behaviour” of the structure, avoiding the activation of local mechanisms. Besides, tie rods are easy to identify on the façade during a survey, which is an important factor when using onsite observation-based approaches. The quality of the “*Façade Connections*” is evaluated through 40 façade configurations, in which different positions and numbers of metal ties are analysed. In the first configuration (C1) no ties are taken into account, while in the while in the configuration (C40) 25% two ties are considered for each floor. The geometry of the metal tie is defined through the average dimensions of the ties detected in the case study, while its stress is evaluated by considering the minimum force that an 80 cm thick irregular masonry wall can resist, assuming the shear failure of the masonry (Podestà and Scandolo 2012). As can be seen in Figure 6.11, the façade configurations presenting tie rods at the upper part of the wall result in higher collapse multiplier values.

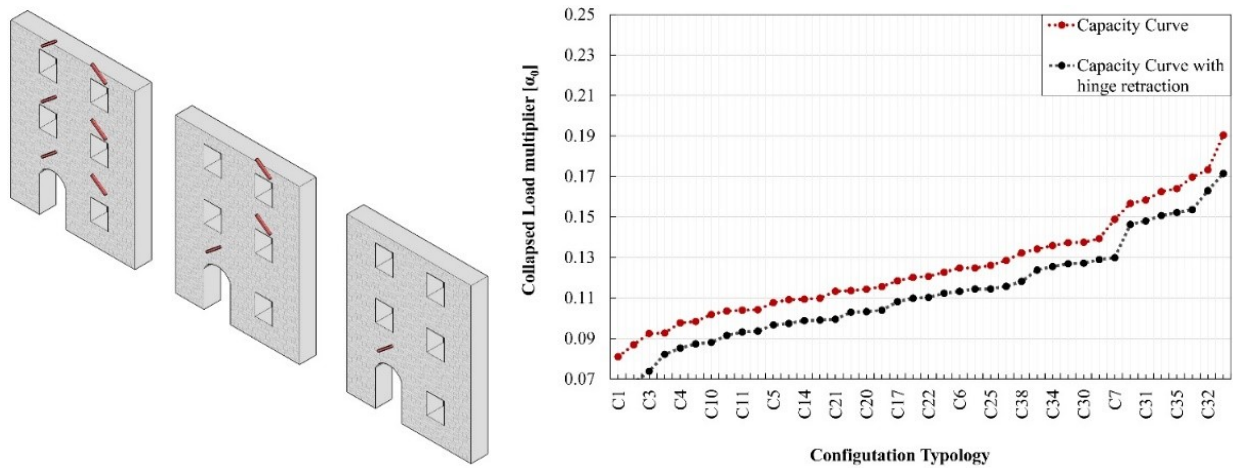


Figure 6.10: Capacity curve of the vulnerability parameter “Façade Connections”.

The damage detected in the case study (Figure 6.11) confirms the results achieved, as well as the importance of connections along all sides of the perimeter walls.



Figure 6.11: Overturning of the wall of the ground floor (SU 112) (a) and detail of the tie of the façade (SU 112) (b, c).

Corner Detachment

In general, the corner of masonry buildings is one of the most critical parts of the structure, as it defines the binding level between two orthogonal walls and, therefore, ensures the global behaviour of the building (Ahmed et al. 2017). In the case of the masonry aggregates, the corner is even more critical: the external SU is not confined by the adjacent SU, which provides a lateral constraint along one side of the building and therefore it is the most stressed building during a seismic event. Vulnerability Index methods, such as (Formisano et al. 2009) and (Vicente 2010), highlight this aspect, assigning the highest score (class D) to the buildings at the end of an aggregate. In this sense, 40 different corner configurations are defined for the vulnerability parameter related to the “*Corner Detachment*”, starting from the façade benchmark. For example, in the first configuration (C1) there are two ties at the top of the corner, the slenderness of the corner is low and there isn't a roof beam. The results show that the collapsed load multiplier is influenced by the slenderness of the corner walls, the dimension and position of the masonry wedge, the presence of ties on the two corners' sides, the presence of a diagonal roof beam at the top, the masonry quality and the regularity of the cornerstones.

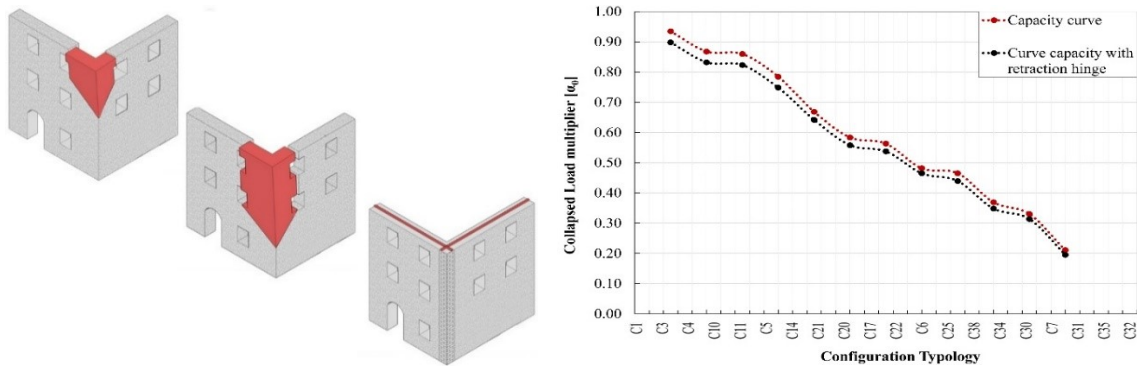


Figure 6.12: Capacity curve of the vulnerability parameter “Corner Detachment”.

The condition of different building corners of the case study after the earthquake agrees with the results obtained (Figure 6.13).



Figure 6.13: Presence of tie rods at the corner of the aggregate (SU196) (a); Corner failure of external buildings of an aggregate, caused by the absence of tie rods (SU 60) (b) and masonry connection between two orthogonal walls (SU 142) (c);

Figure 6.14 shows the capacity curves for different vulnerability parameters analysed. It is important to note that these graphs are essential to identify not the values, but how the trend of the collapse load multiplier changes with the variation of these parameters.

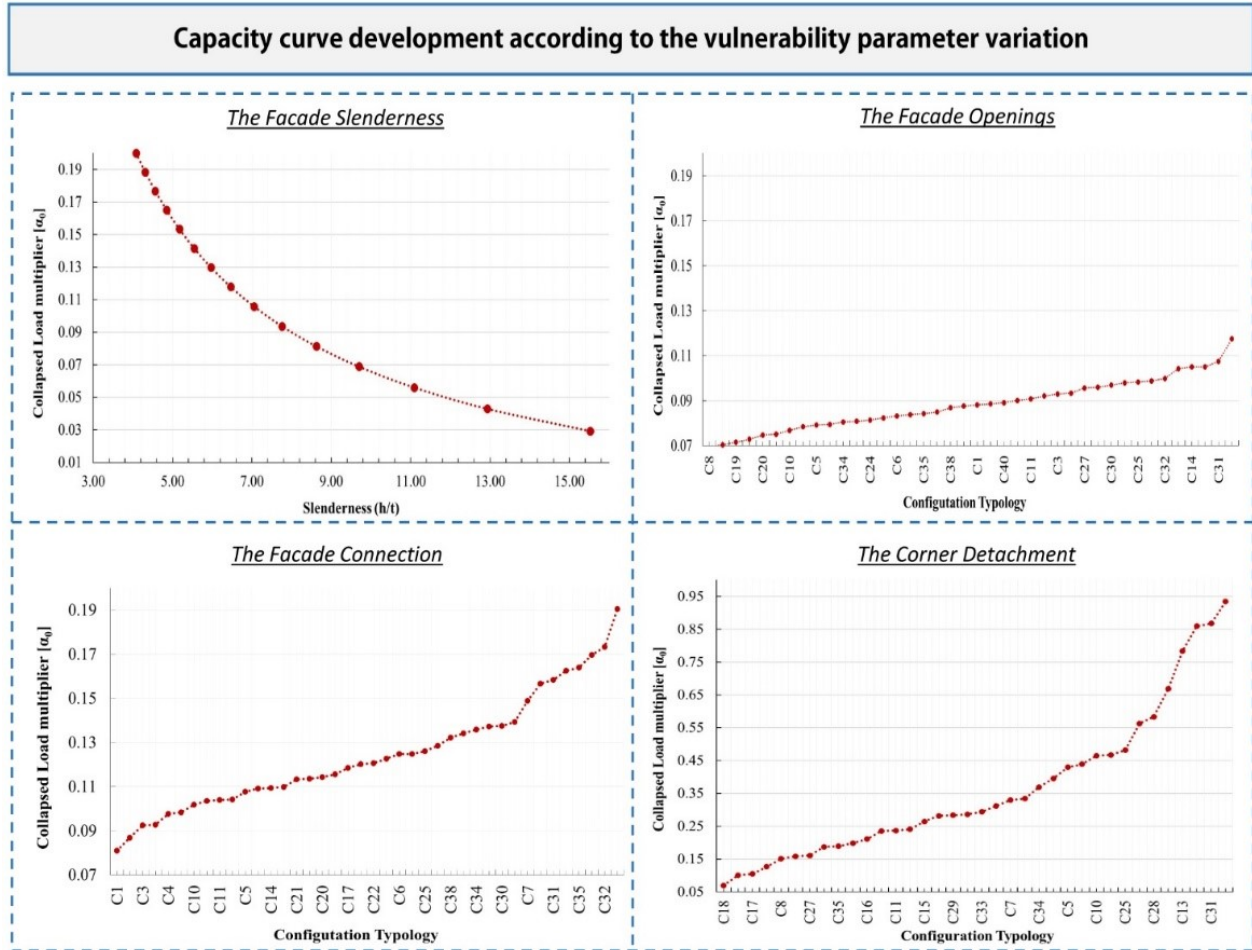


Figure 6.14: Collapsed Load Multiplier evaluation according to the vulnerability parameter variation

6.3.3. The Definition of the vulnerability classes

In the framework of the vulnerability methods analysed herein, the vulnerability level is measured through four classes (A, B, C and D), which grade the condition of the structure to a specific feature or characteristic. Following the parametric analyses performed for the evaluation of capacity curves of each parameter, the vulnerability classes are estimated according to the variation of the collapsed load multiplier. The range of variation of the collapsed load multiplier is divided into four uniform ranges, to which a specific class is assigned. In this way, it is possible to relate each class with a range of façade configurations with specific geometrical and structural features.

As an example, Figure 6.15 shows the flowchart for the classification of the parameter related to the “Façade Slenderness”: wall slenderness values lower than 5 resulted in class A, while the classes B and C correspond respectively to values between 5 and 7, and 7 between and 11. Finally, façades with slenderness values greater than 11 correspond to class D.

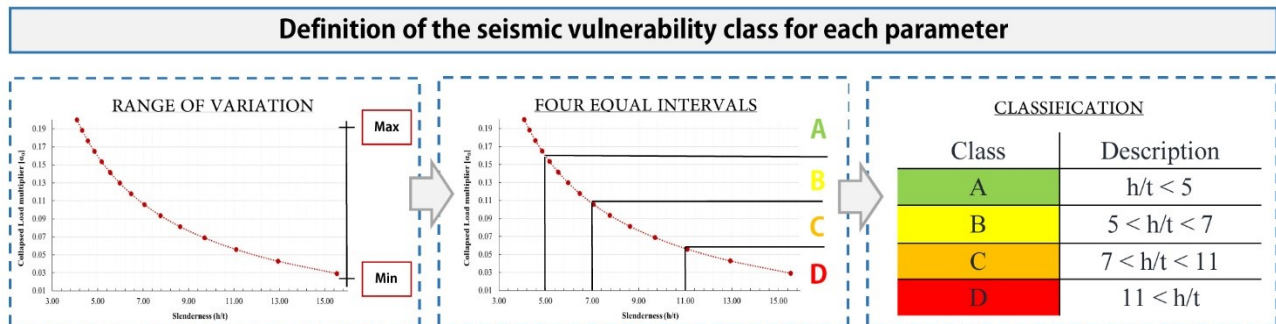


Figure 6.15: Flowchart of the vulnerability class definition for the parameter “Façade Slenderness”.

Moreover, other characteristics of the analysed parameter that can increase or reduce the vulnerability class during the building survey are detailed in specific tables. Since the capacity curve is based on the rigid body limit analysis, other kinds of vulnerabilities are taken into account, such as masonry quality. Past earthquakes have shown that poor masonry quality increased the seismic vulnerability of the buildings, not determining a rigid body behaviour of the façades, but rather destructive collapses (Borghini et al. 2011), (Oyguc and Oyguc 2017), (Borri et al. 2018). According to the double-leaf walls identified in Campi Alto di Norcia, summarised in Chapter 3, two cases are considered for the masonry quality (Figure 6.16): if the façade has a regular wall composition and the masonry is well connected (Case A), the vulnerability class is kept unchanged. On the contrary, if the façade has an irregular wall composition and the masonry is poorly connected (Case B), the evaluated vulnerability class should be penalised in one class. The complete guidelines for the application of the vulnerability parameters of the Improved Method are outlined and presented in *Annex C*.

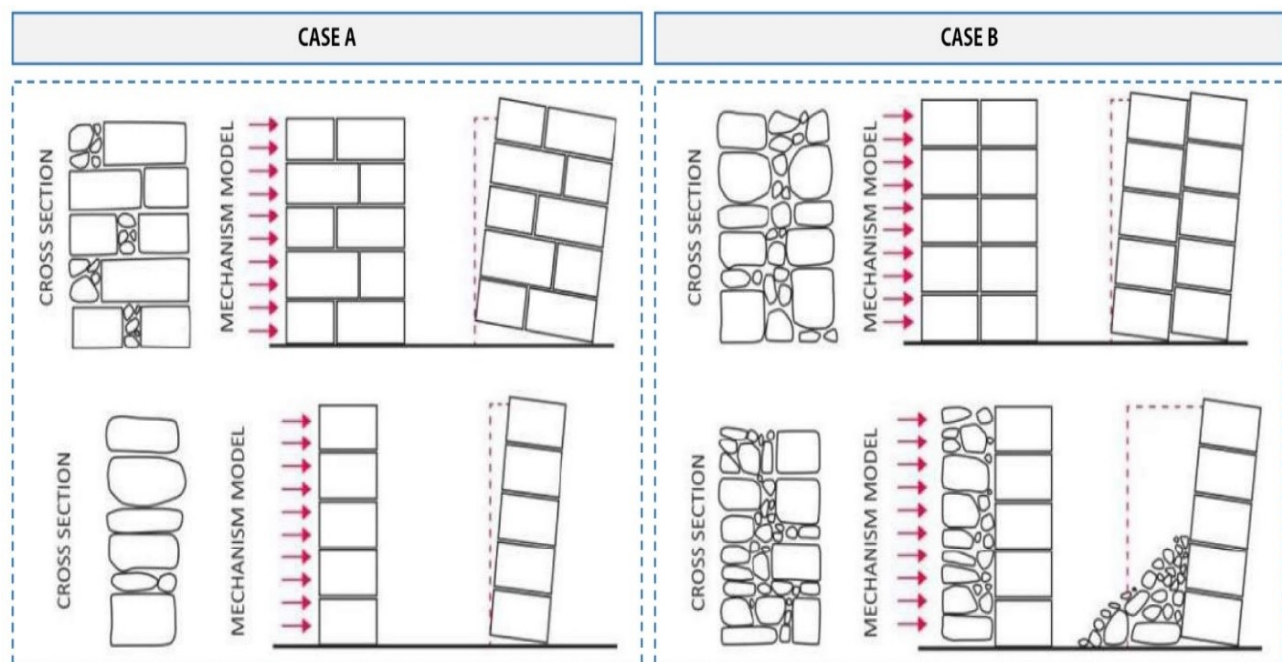


Figure 6.16: Structural behaviour of multiple-leaf stone masonry walls according to the quality of transversal connection among leaves (adapted from Ortega 2018).

The Façade Slenderness

The façade slenderness classification is evaluated for a regular arrangement of the masonry wall in which the double-leaf walls are well connected (case A). When the arrangement of masonry is irregular or when the transversal connection between wall leaves is insufficient (Case B), the slenderness of the observed wall is doubled to take into account of the wall instability (Giuffrè 1990). In the case of high base thicknesses, compared to the average thickness of the façade (i.e., presence of buttresses), the possibility of partial overturning must be considered, and therefore a higher score should be assigned. The score related to each class is the same proposed by (Vicente 2010), i.e., 0 for class A, 5 for class B, 20 for class C and 50 for class D. If the slenderness of the façade is lower than 5, the parameter class can be A, B or C depending on the possibility of an overturning mechanism is triggered and the arrangement of the masonry. When the façade slenderness is within the ranges of 5 to 7 and 7 to 11, the parameter class can be either C or D. Finally, if the slenderness is higher than 11, class D must be assigned regardless the wall composition and the overturning mechanism involved (Table 6.4).

Table 6.4: Seismic vulnerability classes according to the wall slenderness of the façade

Parameter 7: Wall Slenderness				
Slenderness (h/t)	Complete Overturning	Wall Composition	Parameter Class	Score
$h/t < 5$	No	Case A	B	5
		Case B	C	20
	Yes	Case A	A	0
		Case B	B	5
$5 \leq h/t < 7$	No	Case A	C	20
		Case B	D	50
	Yes	Case A	B	5
		Case B	C	20
$7 \leq h/t < 11$	No	Case A	C	20
		Case B	D	50
	Yes	Case A	C	20
		Case B	D	50
$11 \leq h/t$	No	Case A	D	50
		Case B	D	50
	Yes	Case A	D	50
		Case B	D	50

The Façade Openings

The experience acquired during the observation of the damage reported in Chapter 4 allowed adding further aspects that were found to be relevant for the evaluation of the effect of the openings on the global seismic capacity of the SU. In this sense, the analysis performed elaborates on Parameter 10 of the Vicente (2010)'s method, taking into account the local mechanism. The negative influence of the misalignment of the openings,

both vertically and horizontally, is evaluated through the scheme suggested by the Vicente (2010) method and shown in Figure 6.17.

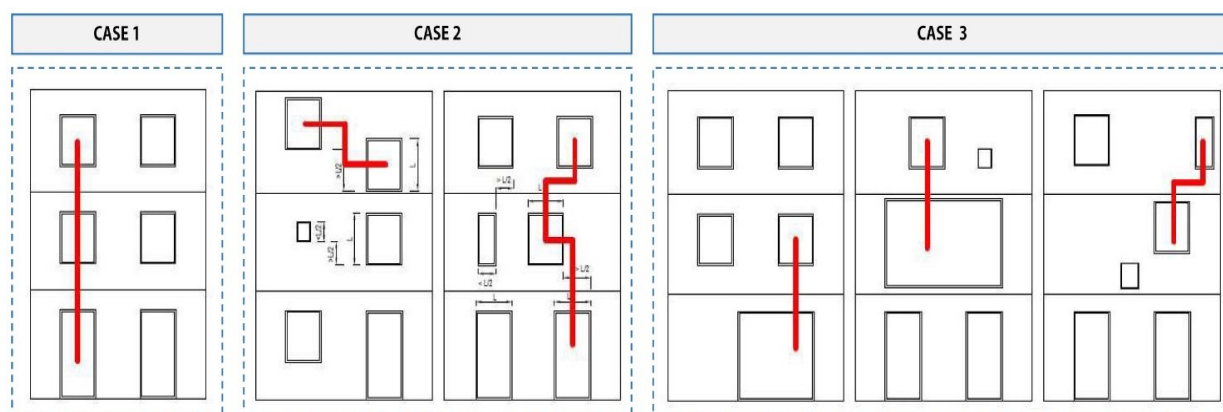


Figure 6.17: Schemes to identify the vulnerability classes, according to the possible configurations of the openings misalignments (adapted from Vicente 2010).

Finally, the value of the score increase according to the wall composition and openings misalignment: class A identifies façades with a percentage of openings lower than 5%, regular wall composition, well-connected masonry and regular or irregular openings disposition (i.e., vertically or horizontally misaligned more than $\frac{1}{2}$ of their width or $\frac{1}{2}$ of their height). The growth of the percentage of openings increases the vulnerability class, according to their misalignment and the wall composition (Table 6.5).

Table 6.5: Seismic vulnerability classes according to the wall slenderness of the façade

Parameter 8: Openings Disposition				
Floor Openings	Wall composition	Openings Misalignment	Parameter Class	Score
Openings < 25%	Case A	Case 1	A	0
		Case 2	A	0
		Case 3	B	5
	Case B	Case 1	A	0
		Case 2	B	5
		Case 3	C	20
25% < Openings < 50%	Case A	Case 1	B	5
		Case 2	B	5
		Case 3	C	20
	Case B	Case 1	C	20
		Case 2	C	20
		Case 3	D	50
50% < Openings < 75%	Case A	Case 1	C	20
		Case 2	C	20
		Case 3	D	50
	Case B	Case 1	D	50
		Case 2	D	50
		Case 3	D	50

The Façade Connections

The effect of the wall connections in the out of plane behaviour of the building is taken into account through the results achieved with the limit analyses, considering the position and the number of ties in the façade. The damage detected in the case study reveals the positive effect of metal ties located on the upper floors of the building. In this sense, only for the vulnerability parameter “*Façade Connections*”, the scores are -20 for class A, -10 for class B, -5 for class C and 0 for class D, to evaluate the vulnerability reduction due to the presence of metal ties (Table 6.6).

Table 6.6: Seismic vulnerability classes according to the wall slenderness of the façade

Parameter 9: Façade connections			
Position of ties in the SU	Number of Ties (x)	Parameter Class	Score
Top Floor	$1 < x$	A	-20
	1	B	-10
Middle Floors (not last neither first floor)	$1 < x$	B	-10
	1	C	-5
First Floor	$1 < x$	C	-5
	0	D	0

Figure 6.18 shows how the vulnerability class should be assigned for the parameter “Façade Connections”, according to the buildings of the case study.



Figure 6.18: The number and distribution of metal ties, highlighted in red, leads to the assignment of class A (a), B (b) and C (c) for the buildings identified.

The Corner Detachment

The evaluation of Parameter 5 is based on four different structural features: the slenderness of the corner walls, the existence of metal ties, the presence of a roof beam at the top of the corner and the masonry quality of the cornerstone (Table 6.7). The damage detected in the case study shows that the presence of the metal tie at one floor doesn't prevent the activation of this mechanism on the upper floors (Figure 6.13). If the wall composition belongs to case B the parameter class assignment has to be reduced by one class. The score related to each class is the same used for all the other parameters (except for the "Façade connections"), i.e., 0 for class A, 5 for class B, 20 for class C and 50 for class D (Table 6.8).

Table 6.7: Structural features for the assignment of the vulnerability class

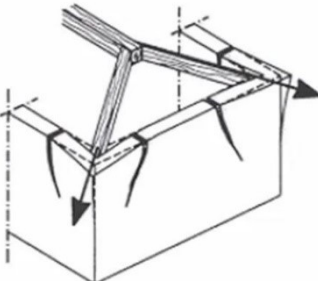
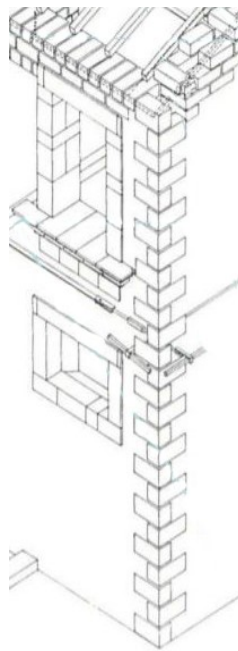
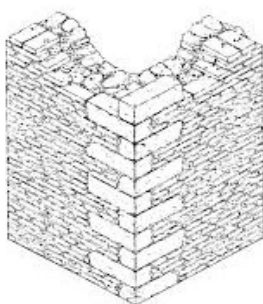
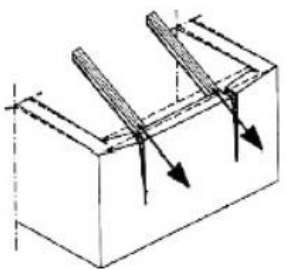
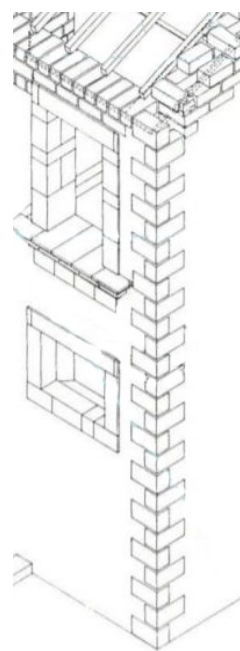
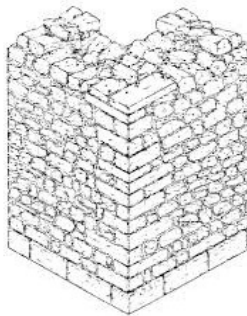
Presence of a Roof beam	Presence of metal ties	Masonry Quality
 Yes	 Yes	 Regular Cornerstone
 No	 No	 No Cornerstone

Table 6.8: Seismic vulnerability classes according to the wall slenderness of the façade

Parameter 10: Corner Detachment					
Wall Slenderness	Roof Beam	Metal Ties	Masonry Quality	Class	Score
$h/t < 5$	NO	YES	Regular Cornerstone	A	0
			No Cornerstone	A	0
		NO	Regular Cornerstone	A	0
			No Cornerstone	B	5

$7 \leq h/t < 11$	YES	YES	Regular Cornerstone	B	5
			No Cornerstone	B	5
		NO	Regular Cornerstone	C	20
			No Cornerstone	C	20
	NO	YES	Regular Cornerstone	B	5
			No Cornerstone	B	5
		NO	Regular Cornerstone	B	5
			No Cornerstone	C	20
	YES	YES	Regular Cornerstone	C	20
			No Cornerstone	C	20
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50
$11 \leq h/t$	NO	YES	Regular Cornerstone	C	20
			No Cornerstone	C	20
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50
	YES	YES	Regular Cornerstone	C	20
			No Cornerstone	D	50
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50

6.4. Definition of seismic vulnerability assessment parameters weight

In the vulnerability index methodology, the weights describe the importance that each parameter have in the seismic vulnerability of the building. These values are derived from the analysis of large amounts of damage data recorded after seismic events (Benedetti and Petrini 1984), (GDNT 1994). As a first assumption, the value of the new weights is defined based on similar parameters present in other already established vulnerability index methodologies, namely in (Ferreira et al. 2017), (Rapone et al. 2018), (Lagomarsino and Podestà 2004) (Table 6.9).

Table 6.9: Distribution of adopted weights

Vulnerability Parameter	Short Description	Rapone et al.	Lagomarsino and Podestà	Ferreira et al.	Value adopted
P1 Façade Slenderness	Ratio between height and thickness and the base of the façade	1.5	1.0	0.5	1.0
P2 Façade openings disposition	Ratio between the openings' area and the total area of the wall, measured at each floor.	1.5	1.0	0.5	1.0
P3 Façade Connections	Presence of metal ties at each floor.	1.5	0.0	2.0	1.2
P4 Corner Detachment	Type of building's corner, considering the presence of ties on the two corners' sides, the masonry quality and the regularity of the cornerstones.	0.0	0.0	0.0	0.5

Table 6.10 summarises the vulnerability parameters, vulnerability classes and corresponding weights, of the improved method.

Table 6.10: Summary table of vulnerability parameters adopted in the Improved Method

Vulnerability Parameters			Vulnerability Class				Weight
			A	B	C	D	
Improved Method (2020)	P1	Quality of the resisting system Construction age, quality of walls' connection	0	5	20	50	1.0
	P2	Conventional strength Walls' shear strength assuming box behaviour	0	5	20	50	1.5
	P3	Number of floors Number of floors in the building	0	5	20	50	1.5
	P4	Horizontal diaphragms Quality of floor type considering stiffness and connection with the walls	0	5	20	50	1.0
	P5	Roof typology Weight and the roof typology (thrust)	0	5	20	50	1.0
	P6	Location of building and type of foundation Topographical conditions of the ground and foundations characteristics and depth	0	5	20	50	0.75
	P7	Façade Slenderness Ratio between the height (H) of the façade and its thickness (b) at the base ($FS = H/b$).	0	5	20	50	1.0
	P8	Façade Openings Ratio between the openings' area and the total area of the wall, measured at each floor.	0	5	20	50	1.0
	P9	Façade Connections Presence of metal ties at each floor.	-20	-10	-5	0	1.2
	P10	Corner Detachment Type of building's corner, considering the presence of ties on the two corners' sides, the masonry quality and the regularity of the cornerstones.	0	5	20	50	0.5
	P11	Aggregate position and interaction Position of the SU in the SA	0	5	20	50	1.5
	P12	Plan configuration Length/ width ratio of the building plan	0	5	20	50	0.75
	P13	Height regularity Mass variation in elevation and the presence of arcades or towers	0	5	20	50	0.75
	P14	Fragilities and conservation state Masonry quality and cracking scenario	0	5	20	50	1.0
	P15	Non-structural elements Type and characteristics of the non-structural elements	0	5	20	50	0.5

Finally, to evaluate the accuracy of the improved method, the latter is adopted in the case study of Campi Alto and compared with the results obtained with Vicente (2010) method.

6.4.1. Application of the improved method

The adoption of the Vicente method (2010) in the case study of Campi Alto di Norcia led to an average normalised I_v equal to 41 and a Standard Deviation (SD) of 11. In terms of distribution, 4% of the evaluated buildings present an I_v index between 10 and 20, 12% between 20 and 30, 30% between 30 and 40, and the remaining 35%, a value between 40 and 50.

The application of the improved method to Campi Alto di Norcia results in an average I_v equal to 44 with Standard Deviation (SD) of 14. Values of the normalised I_v – evaluated according to [2.4] – are between 20 and 30 for the 30% and between 30 and 40, for the 12% of the SUs analysed. The majority of the buildings have an I_v between 40 and 50 (24%) and between 50 and 60 (30%), while the remaining SUs present a seismic vulnerability index value lower than 90. Figure 6.19 and Figure 6.20 compare the scores and the seismic vulnerability of the two methods.

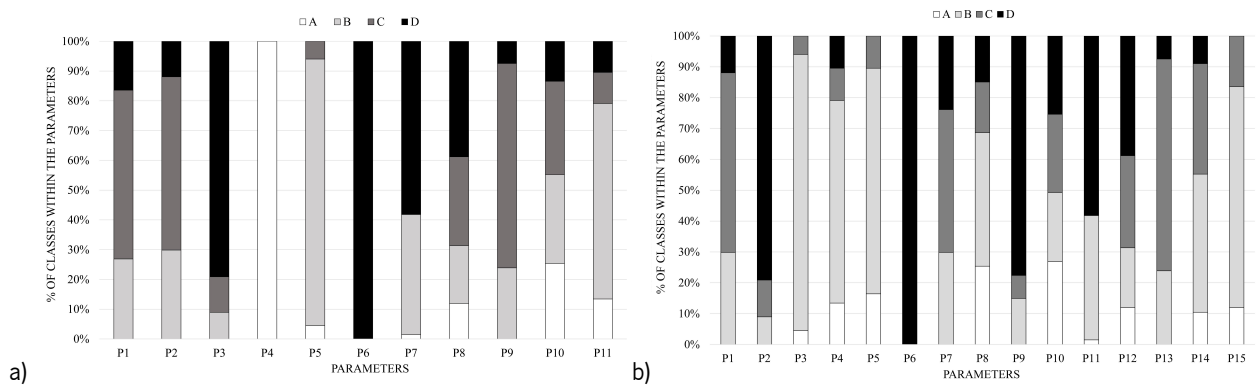


Figure 6.19: Vulnerability classes distribution of the different parameters, according to the Vicente method (a) and the improved method (b).

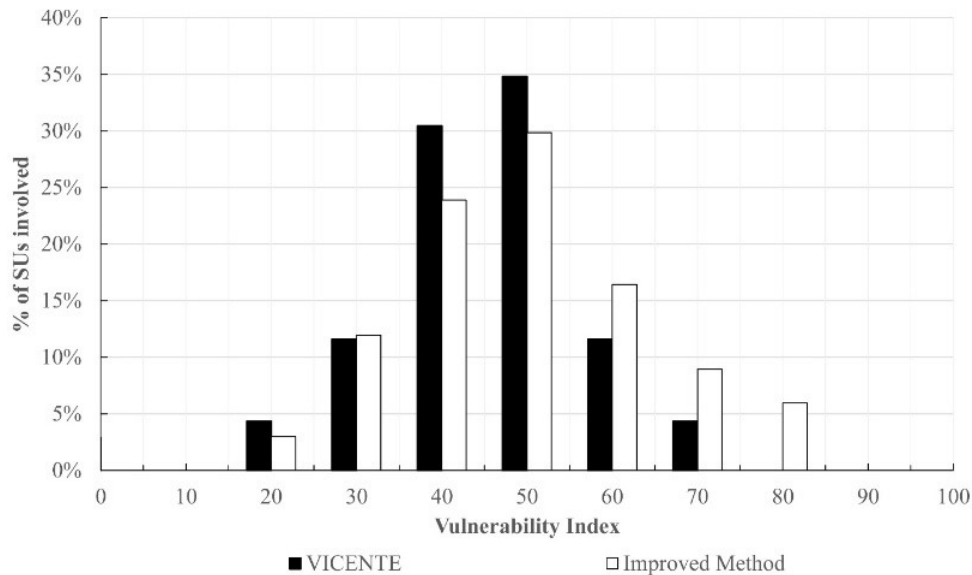


Figure 6.20: Distribution of the I_v through the different methods for 67 SUs.

It is important to note that the Improved method defines new vulnerability indexes for the SUs analysed, increasing the average of the seismic vulnerability of the case study. In the new form, the values are less gathered around a central value – they are, instead, distributed in a broader range (which is reflected by the

higher standard deviation value). Table 6.11 shows the accuracy of the improved by comparing the theoretical damage – obtained applying Equation [5.3] – the real/observed one, observed in loco.

Table 6.11: I_v methods accuracy on the urban scale

I _v Methods	I. Methods accuracy (direct measure)		I. Methods accuracy (assuming a range)	
	Average relative errors	Variance relative errors	Average relative errors	Variance relative errors
Vicente	0.85	0.61	0.58	0.34
Improved Method	0.65	0.49	0.61	0.41

Figure 6. 21 shows the comparison between theoretical and real damage distribution in Campi Alto di Norcia, carried out at building scale by evaluating, for each SU, the distance between the real and analytic damage values. As can be observed, the Improved Method results in better approximations than the Vicente method. However, the improvements still do not capture the actual low (D0) or high (D5) damage classes, so that the estimated damage distribution is concentrated around the middle classes of the EMS-98 scale, even for very low or high vulnerabilities. This leads to an increase of the relative error between the theoretical and real damage, resulting from a sort of incorrect estimation of Bernardini's formulation and from uncertainties of the data used for the vulnerability assessment. To improve the results, the weights of the different parameters of the method are calibrated considering the damage observed for the seismic intensity that hit the construction typology of Campi Alto. Not only the weights of the new vulnerability parameters are calibrated, but all the weights of the Improved method are calibrated to consider the cross-correlation effect between the different parameters.

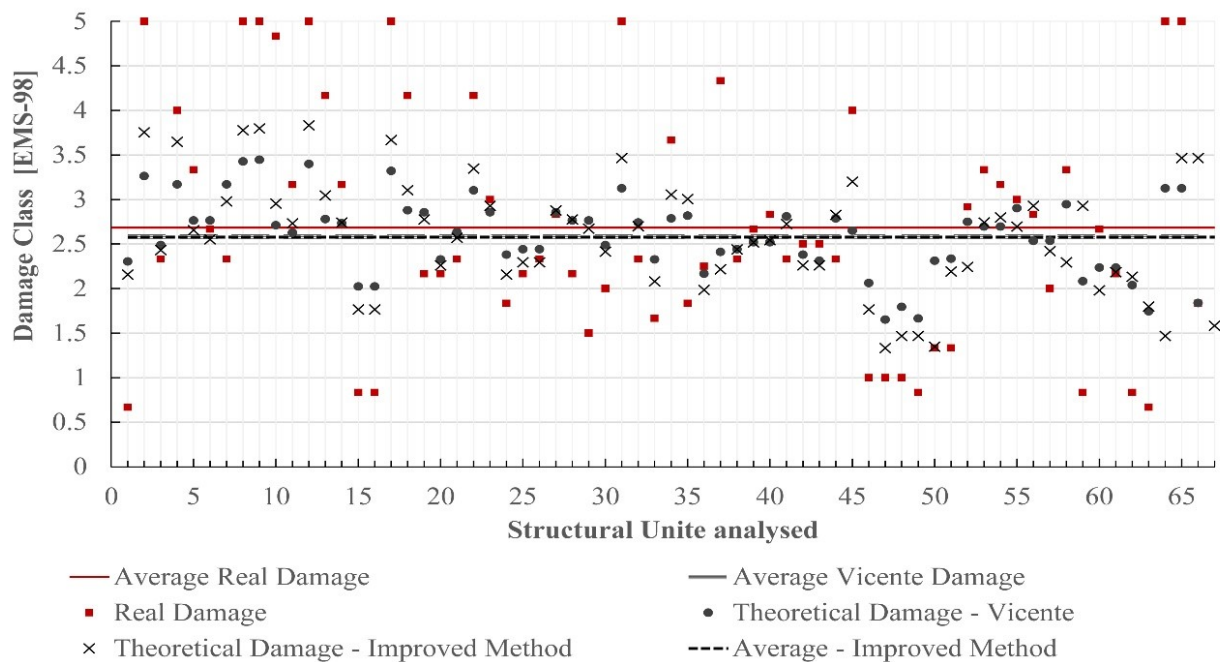


Figure 6. 21: Comparison between the Real Damage detected in the case study and the Theoretical Damage evaluated through Vicente and Improved method for each SUs.

6.4.2. The calibration procedure

To obtain an I_v method more suitable for predicting seismic damage in Campi Alto di Norcia, a calibration procedure is performed to modify the weights associated to the different parameters of the Improved Method, achieving a good agreement between observed and theoretical damage scenario. The calibration is, therefore, focused on the reduction of the relative error (Err) between real (μ_D) and theoretical (μ_T) damage scenario for the i -th building analysed. The exercise consists in relating the distribution of real/observed damages to the vulnerability assessments carried out using parameters with fixed weights and classes, translating – in this specific application – to a problem with 67 linearly independent equations (representing the damage of the buildings surveyed) and 15 unknown variables (the weights used for each building) that cannot be solved at a deterministic level but requires a statistical approach where the evidence of a true state (i.e. damage detected) is expressed in terms of degrees of probability.

A Bayesian linear regression is therefore adopted. The calibration of an I_v method is tackled as an inference problem: the probability of having a particular event E (the damage) given the occurrence of A (the vulnerability) is expressed by Equation [6.5], representing the base for this calibration procedure.

$$P(E_i|A) = \frac{P(A|E_i) \cdot P(E_i)}{P(A)} \quad [6.5]$$

The statistical model combines some *a priori* knowledge of the weights with the observed data points through Equation [6.6]. The weights (θ) were considered as a random vector, following the so-called prior distribution π . After checking the reliability of the first distribution, a posterior distribution $\pi(\theta|y)$ was evaluated through a likelihood function $\pi(y|\theta)$, representing the probability of observing the data (y) given a certain set of weights θ (Lancaster 2004).

$$\pi(\theta|x) = \frac{\pi(x|\theta)\pi(\theta)}{\pi(x)} \quad [6.6]$$

The posterior distribution represented then an update of the prior one correcting the initial estimation. The Markov Chain Monte Carlo simulation sampling method (MCMC) was used for computing the solution. Following an iterative process, a proposed new distribution was accepted or rejected until the solution converged within a given probability (Haario et al. 1999). Figure 6.22 summarises the procedure followed to define the distribution of the weights that best describe the real damage scenario in the case study.

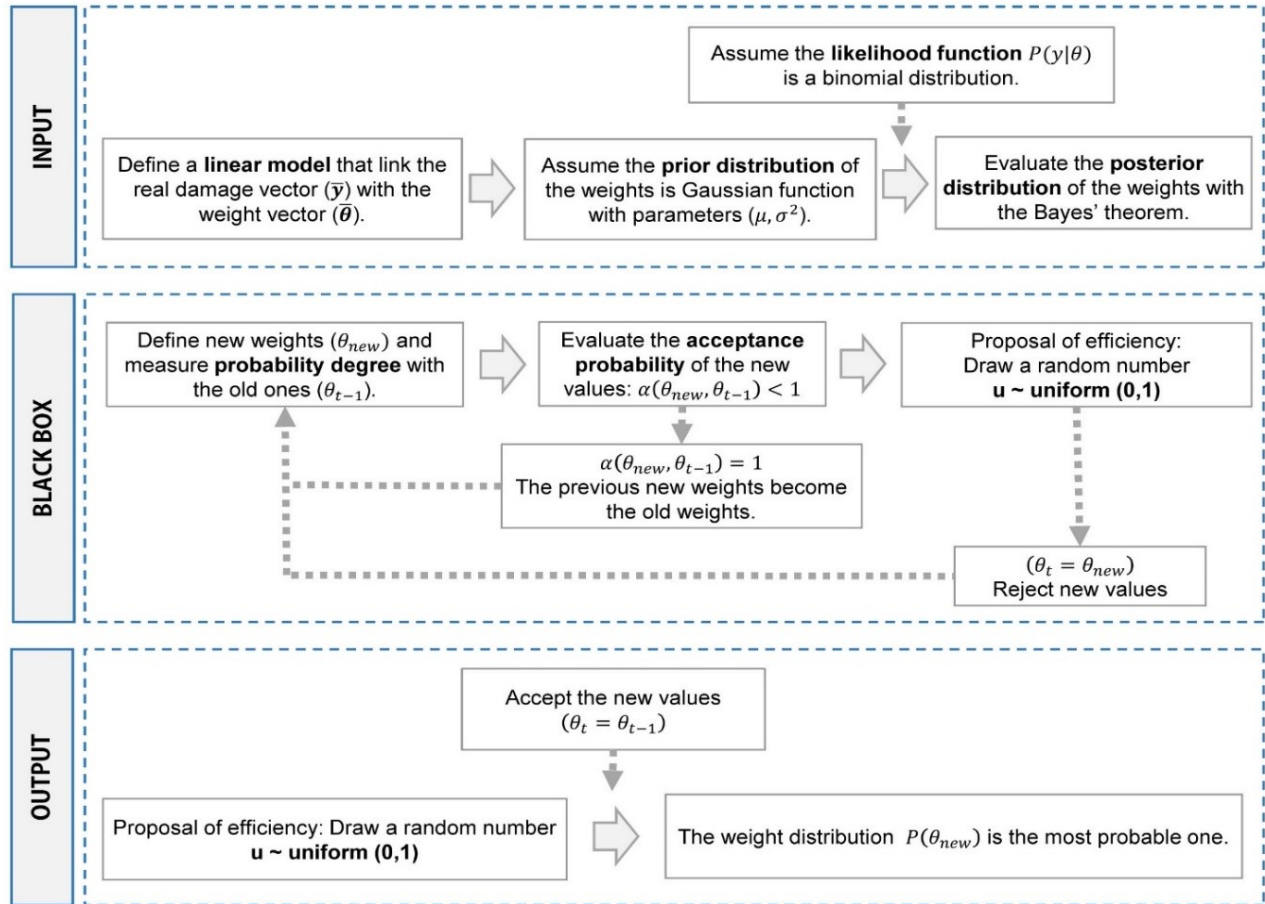


Figure 6.22: Procedure followed for the identification of the weight distribution.

The analysis of the correlation between the weight distributions and the real damage detected, and its subsequent improvement is made using a MATLAB-based software module called UQLab (Marelli and Sudret 2014). Figure 6.23 and Table 6.12 compares the vulnerability index distribution, considering the Vicente method, the non-calibrated Improved Method and the Improved method after the calibration. As can be seen in Table 6.12, the average and standard deviation of the normalised I_v of the case study tend to increase with the calibration, reducing the concentration of vulnerability indexes around the central value of the distribution.

Table 6.12: Average normalised I_v according to the different methods analysed.

Analysis	I_v Methods	Average	Standard Deviation (SD)
Original Method	Vicente	41.0	11.2
Not Calibrated	Improved method	44.2	13.8
Calibrated	Improved Method	45.4	20.3

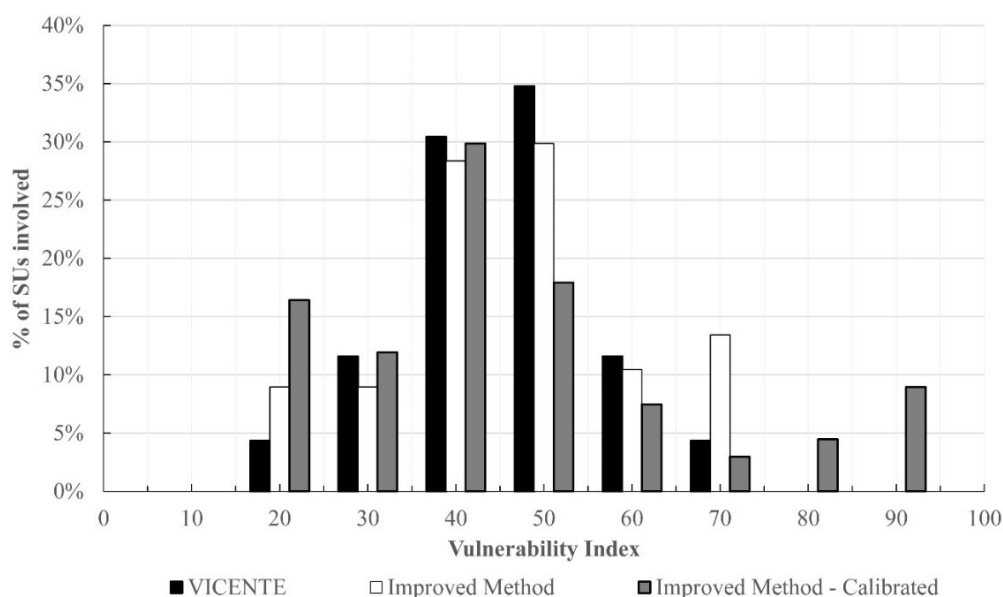
Figure 6.23: Distribution of the I_v through the different methods for 67 SUs.

Table 6.13 and Figure 6.24 compare the average relative errors between the Vicente method, the non-calibrated Improved Method and the calibrated one. A better accuracy of the Improved Method is achieved, reducing the mean relative error between real and theoretical damage from 0.85 to 0.48, considering the direct measure (i.e. each SU individually with no uncertainty). Since the calibrated I_v values were less concentrated around an average value and more distributed along the damage curve, the SD did not suffer for strong modifications.

Table 6.13: I_v methods accuracy of the methods analysed

Analysis	I. Methods	Average relative errors	Variance relative errors
Original Method	Vicente	0.85	0.61
Not Calibrated	Improved method	0.65	0.49
Calibrated	Improved Method	0.48	0.38

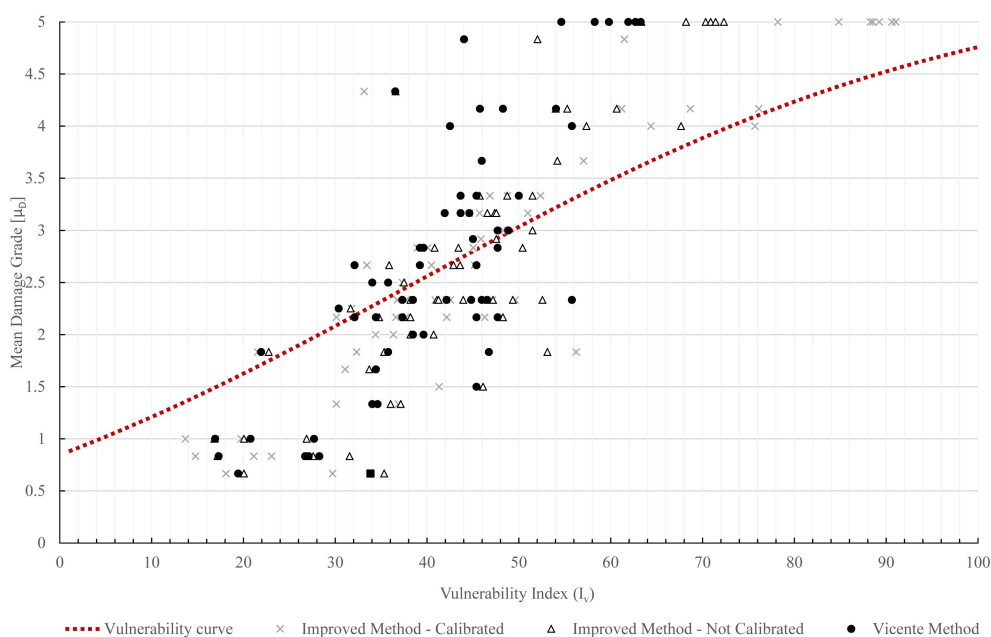


Figure 6.24:: Comparison between the Vicente method, the Improved Method not calibrated and calibrated.

Table 6.14 compares the calibrated and non-calibrated weights used in the Improved method for vulnerability assessment and damage scenario evaluation.

Table 6.14: New calibrated weights for Vicente method.

Calibrated weight for the parameters of the Vicente method based on damage survey in Campi Alto di Norcia			
Parameters		Not Calibrated	Calibrated
P1	Quality of the resisting system	1.00	2.00
P2	Conventional strength	1.50	2.00
P3	Number of floors	1.50	0.50
P4	Horizontal diaphragms	0.75	3.00
P5	Roof typology	1.50	1.00
P6	Location of building and type of foundation	0.75	0.75
P7	Façade Slenderness	0.75	3.00
P8	Façade Openings	1.00	0.75
P9	Façade Connections	1.00	0.50
P10	Corner Detachment	1.00	2.00
P11	Aggregate position and interaction	0.50	1.00
P12	Plan configuration	1.00	0.75
P13	Height regularity	1.00	0.75
P14	Fragilities and conservation state	1.17	1.00
P15	Non-structural elements	0.50	1.00

6.4.3. The expert opinion

The relative importance of the vulnerability parameters that compose the Improved Method is also examined here based on expert opinion. This aims at overcoming the limitations of the previously presented calibration procedure. The answers collected are analysed using the Analytic Hierarchy Process (AHP) Method (Saaty 1980), based on the pairwise comparison of parameters, carried out by assigning an intensity of importance, on a scale from 1 to 9, where 1 means that two parameters have equal significance and 9 means that one factor has significantly higher importance than the other (*Table 6. 15*). The aim of this method is to identify the parameters that are more relevant and to define an order of magnitude of the value of the weights.

Table 6. 15: Scale for the evaluation of the intensity importance (Saaty 1980)

Intensity of importance	Definition	Explanation
1	Equal Importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favour one parameter over another
5	Essential or strong importance	Experience and judgment strongly favour one parameter over another
7	Extreme importance	The evidence favouring one parameter over another is of the highest possible order of affirmation
2, 4, 6, 8	Intermediate values between two adjacent judgements	When compromise is needed

The AHP results in an $n \times n$ matrix, in Equation [6.7], where n is the number of parameters, and the relative importance is defined by the judgment of the pairwise comparison $a_{ij} = w_i/w_j$, for all decision elements and their reciprocal, $a_{ji} = 1/a_{ij}$.

$$A = \begin{pmatrix} w_1/w_1 & w_1/w_2 & \cdots & w_n/w_1 \\ w_2/w_1 & w_2/w_2 & \cdots & w_n/w_2 \\ \vdots & \vdots & \ddots & \vdots \\ w_n/w_1 & w_n/w_2 & \cdots & w_n/w_n \end{pmatrix} = [a_{ij}] \quad [6.7]$$

The relative weights for each judgement are defined by the solution of the linear Equation [6.8], where $\mathbf{W}$ is the normalised right eigenvector ($W = \{w_1, w_2, \dots, w_n\}^T$) and λ_{max} is the corresponding largest eigenvalue of the square matrix $\mathbf{A}$.

$$\mathbf{A}\mathbf{W} = \lambda_{max}\mathbf{W} \quad [6.8]$$

It is shown, that in the case of a reciprocal matrix perfectly consistent of order n , $\lambda_{max} = n$. In this case, it is not necessary to fill all the elements of the matrix and it is sufficient to define $(n - 1)$ elements, as long as these are arranged in order not to be dependent. On the contrary, for a not perfectly consistent matrix it is: $\lambda_{max} > n$. (Saaty 1980) proposes the Consistency Ratio (CR), in Equation [6.9], to evaluate the level of consistency of the matrix $\mathbf{A}$, defined as the ratio between the Consistency Index (CI) [6.10] and the Random Consistency Index (RCI) (Table 6. 16).

$$CR = \frac{CI}{RI} \leq 0.1 \quad [6.9]$$

$$CI = \frac{(\lambda_{max} - n)}{n - 1} \quad [6.10]$$

Table 6. 16: Evaluation of the RCI according to (Saaty 1980).

n	3	4	5	6	7	8	9	10	11	12	13	14	15
RCI	0.58	0.90	1.12	1.24	1.32	1.41	1.45	1.49	1.51	1.48	1.56	1.57	1.59

The input data for the definition of the matrix $\mathbf{A}$ [6.7] is provided by a questionnaire, developed in Google Forms and distributed to more than 200 experts. Since the matrix $\mathbf{A}$ is reciprocal, the number of judgments required should be $n(n-1)/2$ and the survey form should be composed of 105 questions to consider 15 parameters.

To save the time of the participants and to avoid repetitive correlations, if the judgments are perfectly consistent [6.9], the number of questions can be reduced, according to the formulation [6.11]:

$$a_{ij} = a_{ik} \cdot a_{kj} \quad \forall i, j, k \quad [6.11]$$

Harker and Vargas (1987) propose to evaluate the missing judgments through the geometric mean of the values, obtained with different k . This process allows to complete all the correlations within the parameters and to reduce the survey form: 50 questions are prepared, as a compromise between the maximum $(n(n-1)/2)$ and minimum $(n - 1)$ number of questions required. The complete questionnaire for the definition of the matrix **A** is outlined and presented in *Annex D*.

The experts are selected in the professional and academic environment, such as professors, masters' degree, PhD and postgraduate students, that are involved in the field of seismic engineering and heritage conservation. To have a better understanding of the level of expertise of each participant, the participants are asked to grade their experience and level of expertise on a scale of 1 to 5. Figure 6.25 shows that the majority of the participants are PhD researchers (53%) and Professionals (26%), with a valuable contribution of Master's Students (11%), Postgraduate researchers (5%) and Professors (5%). The level of expertise of the participants is on average between moderate (44%) and high (33%). 11% of the interviewed define himself with a minimal or very high level of expertise in the subject.

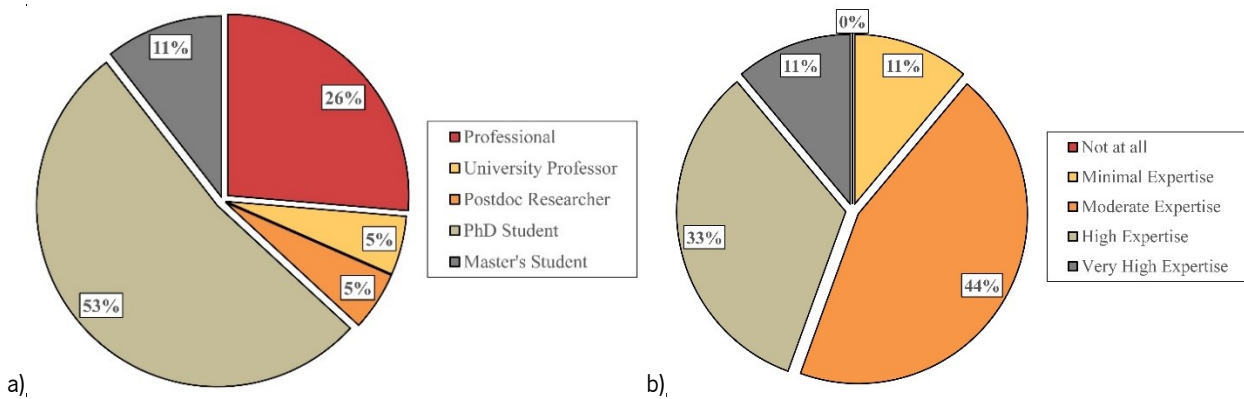


Figure 6.25: Professional position (a) and level of expertise of the participants (b).

For each questionnaire, the matrix **A** and the corresponding eigenvalues (λ_{max}) and eigenvector (**W**) are evaluated, considering the consistency ratio of the post-processed data. Finally, the average value of the relative weights is obtained from all the answers (Table 6.17).

Table 6.17: Average of all the weight values evaluated for each questionnaire through the AHP.

Weight values based on the expert opinion			
Parameters		Average	Standard Deviation
P1	Quality of the resisting system	0.097	0.015
P2	Conventional strength	0.162	0.020
P3	Number of floors	0.092	0.016
P4	Horizontal diaphragms	0.103	0.011
P5	Roof typology	0.056	0.008
P6	Location of building and type of foundation	0.049	0.006
P7	Façade Slenderness	0.080	0.008
P8	Façade Openings	0.072	0.006
P9	Façade Connections	0.053	0.006
P10	Corner Detachment	0.028	0.003
P11	Aggregate position and interaction	0.054	0.007
P12	Plan configuration	0.036	0.004
P13	Height regularity	0.039	0.007
P14	Fragilities and conservation state	0.052	0.005
P15	Non-structural elements	0.027	0.009

The results show that, according to the experts' opinion, the vulnerability parameters can be ordered from the most to the less relevant as follows: P1 (Quality of resisting system), P2 (Conventional strength), P3 (Number of floors), P4 (Horizontal diaphragms) and P7 (Façade Slenderness). On the contrary, according to the experts, the less relevant parameters are P6 (Location of building and type of foundation), P10 (Corner detachment), P12 (Plan configuration), P13 (Height regularity), P14 (Fragilities and conservation state) and P15 (Non-structural elements). Regarding the other parameters, the level of importance is in the average medium. Figure 6.26 summarises the distribution of the vulnerability parameters, according to their relative importance.

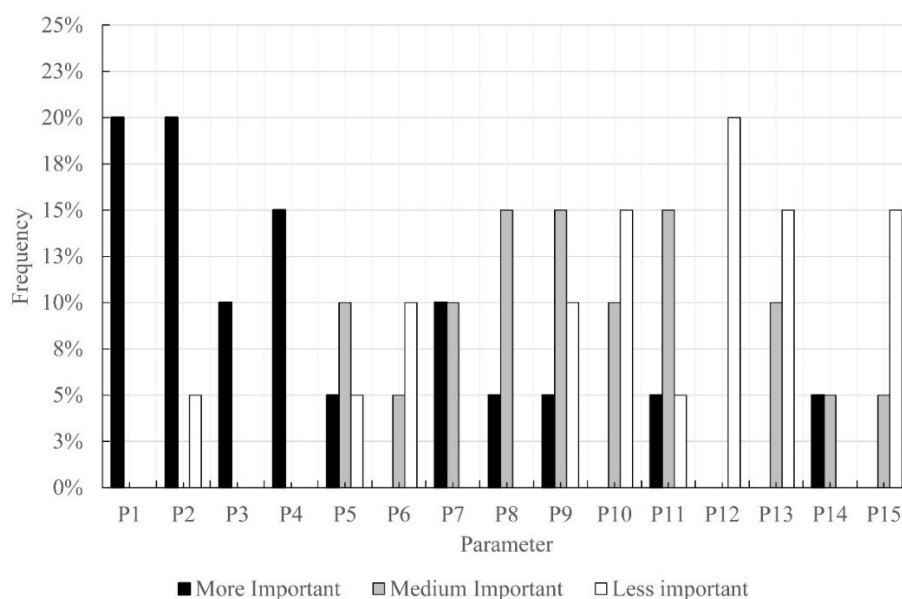


Figure 6.26: Frequency of the parameters according to the importance defined by the experts.

6.4.4. Comparison of the results

The answers to the questionnaire show the level of importance assigned by the experts to the 15 parameters that form the Improved Method. In this way, it is possible to get an idea of the contribution that different elements have in the seismic vulnerability of masonry buildings in aggregate, based on the experience gained in the field of research and from the damage observation of historic centres after earthquakes. Since the weights' values of the Improved Method were calibrated based on damage data for a single seismic intensity, the results achieved show the general importance of the parameters according to the experts, regardless the seismic intensity. To compare the results obtained from the numerical analysis with the mean eigenvector evaluated from all the answers with the AHP, it is necessary to multiply the latter by a real positive number. Figure 6.27 shows the comparison between the values of the weights defined with the numerical calibration and the ones based on the expert opinion.

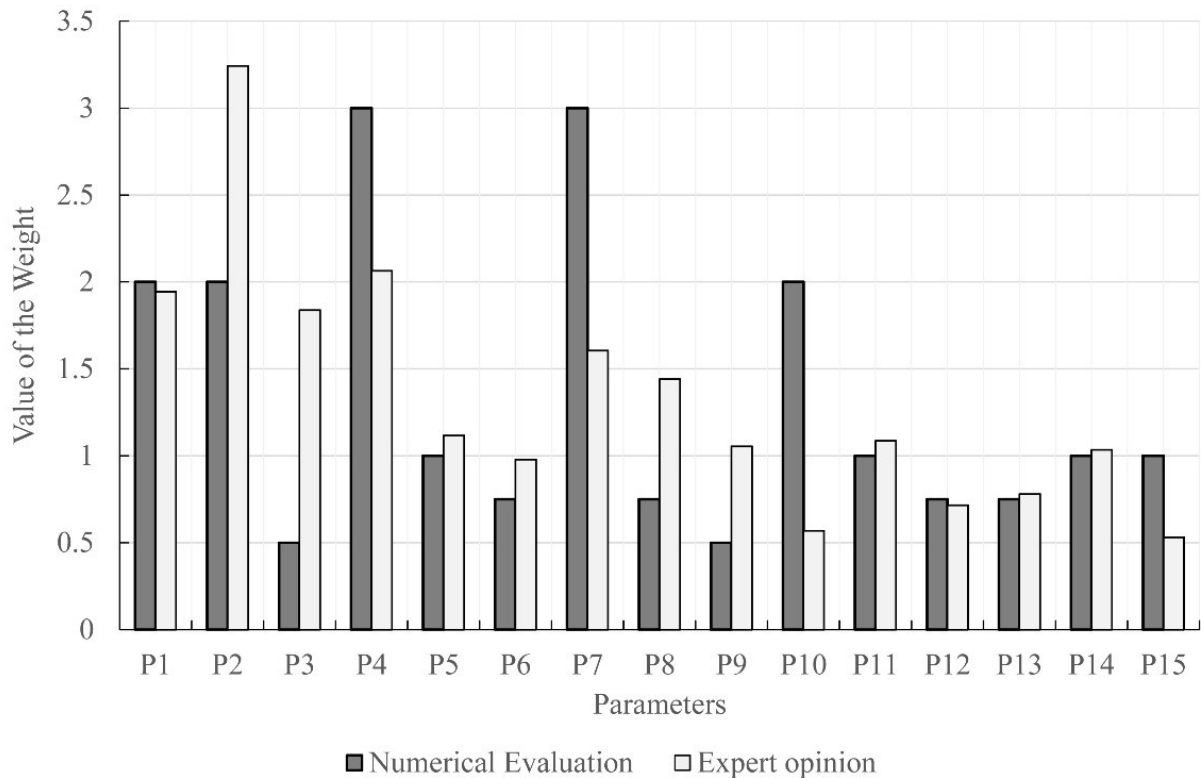


Figure 6.27: Comparison of the weights evaluated with the expert opinion and numerical analysis.

In some case, namely for Parameters P1, P5, P6, P11, P12, P13 and P14, the value of the weight obtained from expert-opinion and numerically is similar. However, there is a notable difference for other parameters, for example for P3 (number of floors), P7 (façade slenderness) and P10 (corner detachment). Despite these variations, it is worth noting that the expert opinion-based values agree with the results of the calibration about the importance of some fundamental parameters, such as the quality of the resisting system, the conventional strength and the horizontal diaphragms.

6.5. The Information Quality Index (IQ)

The individual building's vulnerability is influenced by the knowledge of the uncertainties related to considered parameters. The application of the Improved Method to the case study of Campi Alto is therefore repeated in the following introducing an *Information Quality* (IQ) index, which considers a Confidence Level (CL) defined for each one of the different parameters considered in the I_v method. A 4-level confidence scale is developed based on the experience gained during the damage survey:

- *High confidence level* (H): high-quality information coming from complete (internal and external) surveys and direct measures of the buildings, including structures, materials, details, etc. High confidence in the recorded data.
- *Medium confidence level* (M): medium-quality information coming from external surveys of the buildings and information obtained from historical/critical analysis or visual inspections. Medium confidence in the recorded data, which are obtained through direct/indirect measurements.
- *Low confidence level* (L): based on the external geometrical survey of the building, together with considerations backed on consolidated knowledge for typologically similar buildings. Very low confidence in the recorded data, which mainly result from hypotheses not verified for the building or the urban setting.
- *Absent confidence level* (A): information adopted with limited confidence or by random choice. The attribution of the parameters' vulnerability class is merely indicative.

Three additional intermediate categories are further introduced, to reduce the arbitrary choices of a class and provide an improved score assignment (Ferreira et al. 2020): High-Medium (HM), Medium-Low (ML) and Low-Absent (LA). The definition of a large number of confidence classes is related to the specific features of the constructions, characterised by high variability, and by the experience of the surveyor, that, as said, highly influences the outputs of I_v methods.

Figure 6. 28 shows the distribution of the IQ Index in the case study. Thanks to the different information collected and the great detail of inspections performed, more than 66% of the parameters resulted in a high – medium CL.

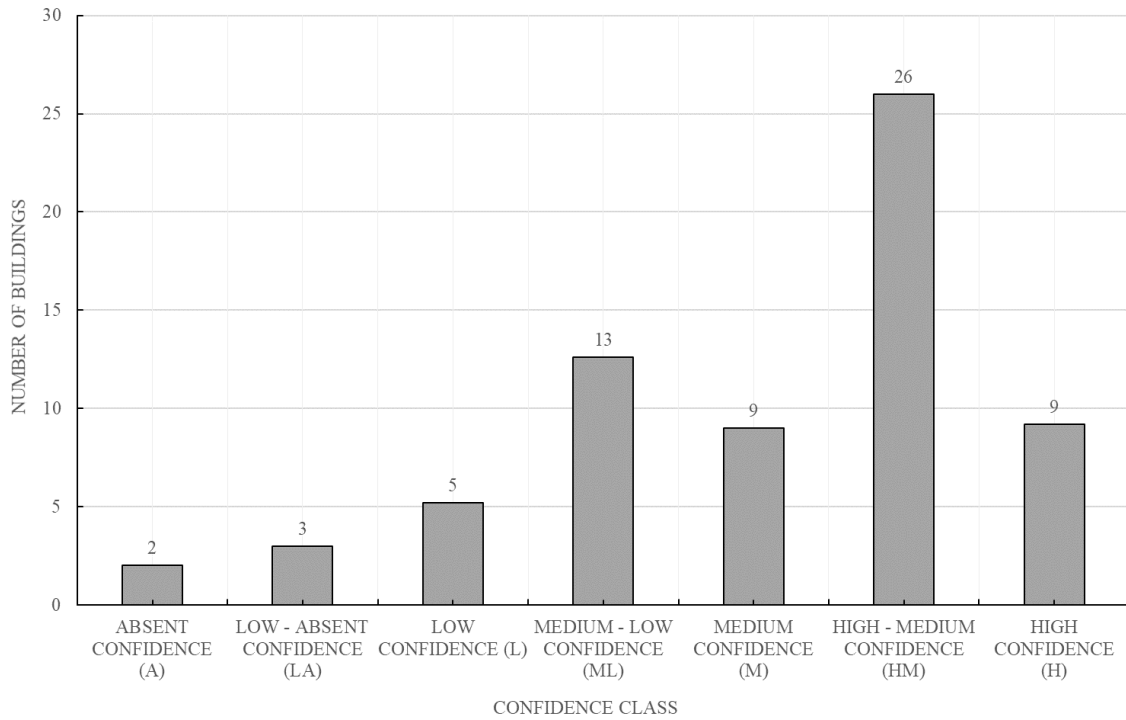


Figure 6. 28: Distribution of the Confidence Classes in the case study.

Regarding the assessed CL, a coefficient accounting for related uncertainties (UL) is determined (Table 6.18). The final CL for each class results then from the weighted average of the different CL ranges assigned to the buildings, neglecting the extreme cases where damage is equal to D0-D1 or D5 according to EMS-98 classification.

Table 6.18: Uncertainty ranges and definition of the confidence classes.

Confidence Level (CL)	Uncertainty Level (UL)
High Confidence (H)	0.0 %
High - Medium Confidence (HM)	10.0 %
Medium Confidence (M)	20.0 %
Medium - Low Confidence (ML)	30.0 %
Low Confidence (L)	40.0 %
Low - No Confidence (LA)	50.0 %
No Confidence (A)	60.0 %

The uncertainty level (UL) is applied to the weight of each parameter, determining the *relevance* (Δ) of the parameter for the seismic behaviour of the building, Equation [6.12].

$$\Delta = \frac{\sum p_i \cdot UL}{\sum p_i} \quad [6.12]$$

Where p is the weight of the i -th parameter and UL the corresponding uncertainty given in Table 6.18.

The Δ value is used to increase or reduce the I_v defining a range where the vulnerability index expresses a correct result [6.13]. These bounds can be seen as the transposition of the percentage of uncertainty coming from the evaluation of the vulnerability parameters. The range of numbers associated to each class is evaluated by checking whether the average of the observed damage ($\bar{\mu}_R$) is within the upper (μ_T^+) and lower bound (μ_T^-) of the analytical theoretical damage, assessed by increasing and reducing the I_v . The definition of the CL to be assigned to each class is defined through a step-by-step procedure, in which the uncertainty interval is calculated after assessing each building's vulnerability, see Figure 6.29.

$$I_v = \begin{cases} I_v^- = \sum p_i(1 - \Delta) \cdot c_i \rightarrow \mu_T^- & (\text{lower bound}) \\ \bar{I}_v = \sum p_i \cdot c_i \rightarrow \bar{\mu}_T \\ I_v^+ = \sum p_i(1 + \Delta) \cdot c_i \rightarrow \mu_T^+ & (\text{upper bound}) \end{cases} \quad [6.13]$$

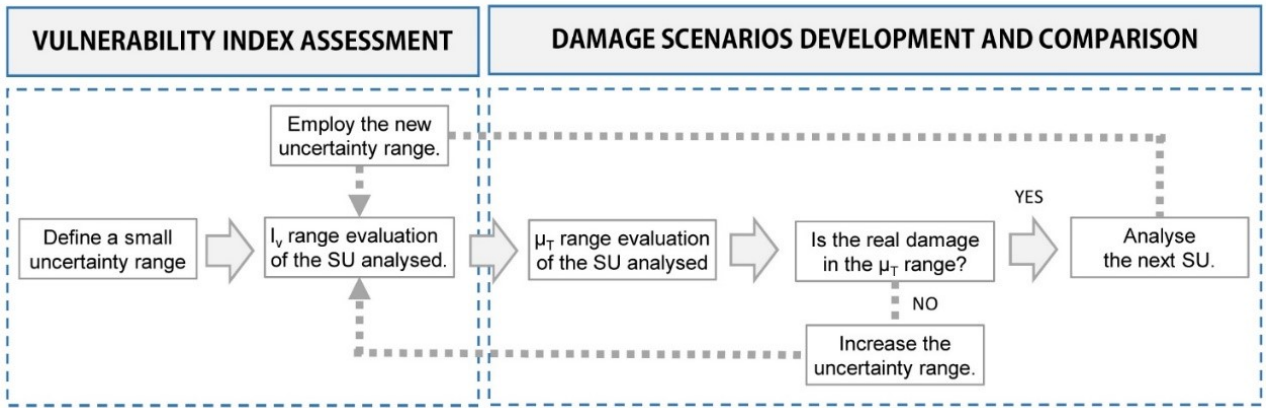


Figure 6.29: Step by step procedure for the LC definition of each confidence class.

The use of the IQ index in the assessment of the seismic vulnerability provides a range of possible I_v values for each of the building analysed. In this case study, the uncertainty classes used to describe the knowledge of the different I_v parameters, belong, on average, to the Medium Confidence Class.

When considering the upper and the lower bound I_v values, a new damage scenario was defined for the case study hit by the seismic intensity $I_{EMS98} = 8$ (Figure 6. 30 and

Figure 6. 31). The line defining the average of the real damage overlapped the line of the mean theoretical damage “Average Calibrated”, thanks to the implemented weights’ calibration procedure. In this sense, the upper and lower bound allowed better to evaluate the expected damage of the single SU, while on average define the possible damage scenarios of the case study according to the EMS98 scale. Also, in this case, the limitation of the mean damage formulation, Equations [2.3] and [2.4], in estimating high or low damage classes

for different vulnerability values was evident: while real damage values are more scattered, the distribution of the estimated damage is concentrated around the average values.

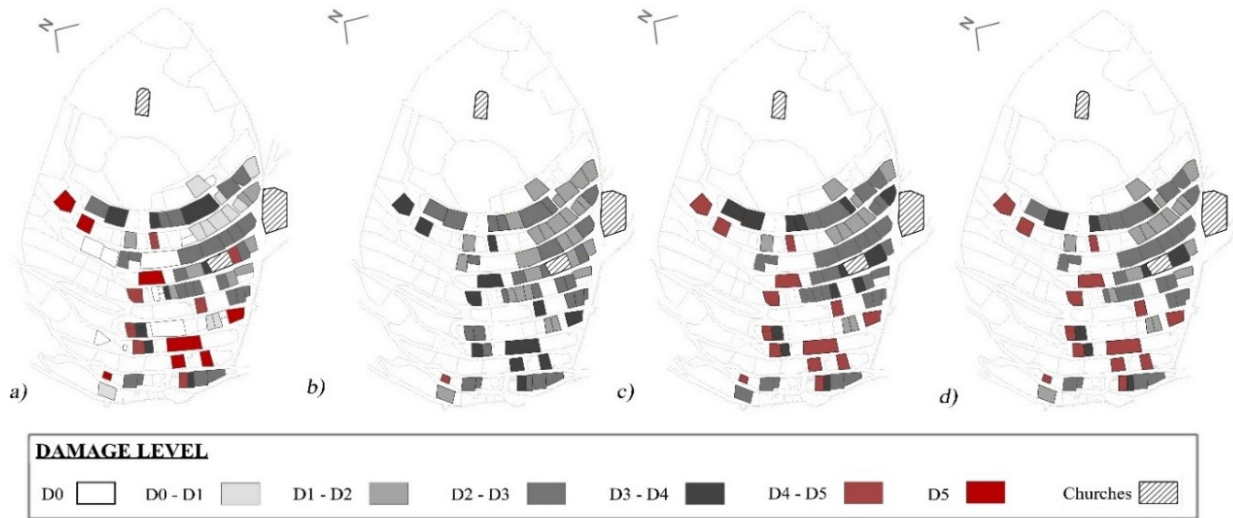


Figure 6. 30: Comparison between the real damage detected in the case study (a) and the theoretical damage scenarios evaluated through the Calibrated improved method ($-IQ$) (b), Calibrated improved method ($+IQ$) (c) Calibrated improved method matching ($-IQ$) and ($+IQ$) (d) for each SU.

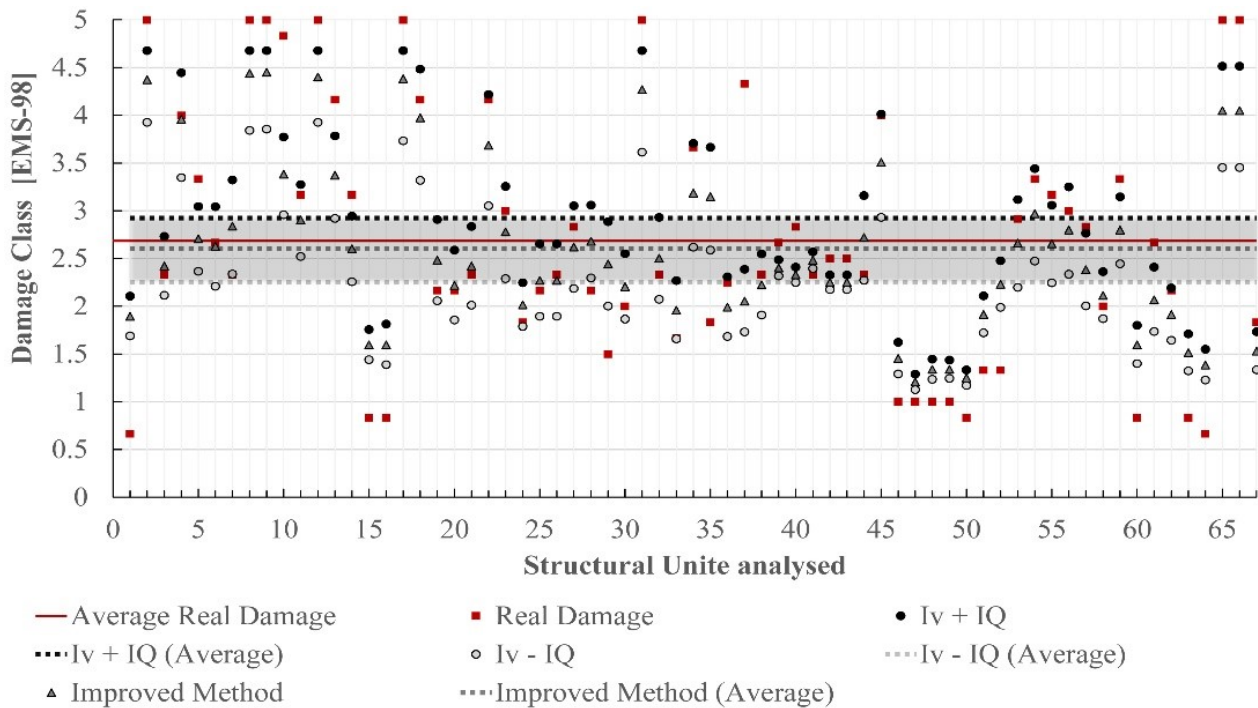


Figure 6. 31: Comparison between Real Damage and the Analytical Damage evaluated through the calibrated method considering the IQ Index. The grey area defines in average the possible damage scenarios of the case study according to the EMS98 scale.

By considering the average vulnerability values obtained for the calibrated Improved Method and changing the macroseismic intensity, it is possible to numerically develop various damage scenarios for the urban centre of the case study analysed. The vulnerability curves presented in Figure 6. 32 show the possible damage scenarios

for different macroseismic intensities. The curve, representing the real damage of the case study for different seismic intensity, is obtained from the average vulnerability value corresponding to different scenarios estimated through Equations [2.3] and [2.4]. The average I_v for the calibrated Improved Method is 45.4, while the mean values for the upper bound (+ IQ) and lower bound (-IQ) is equal to 53.2 and 37.2.

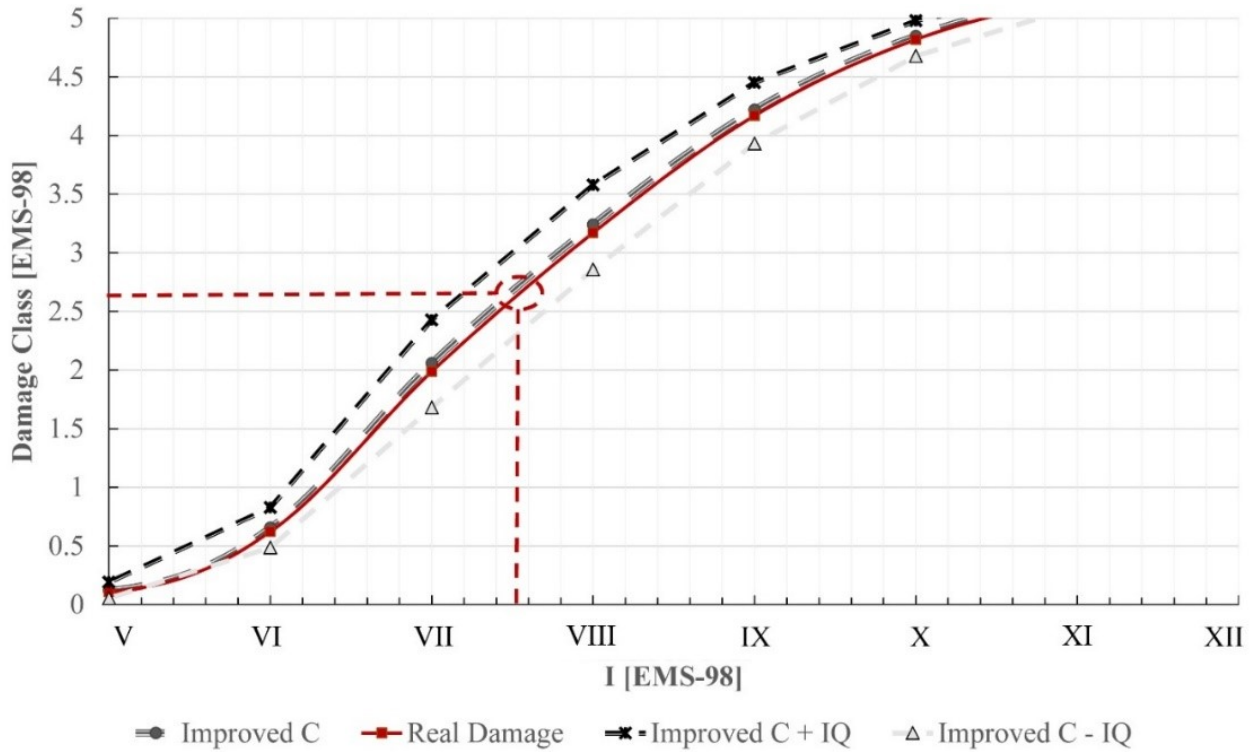


Figure 6. 32: Vulnerability curves of the case study for different seismic intensities using the calibrated improved method (C) considering the upper and the lower bound I_v values with the IQ ; in red the damage and the intensity of the seismic events of October 2016 is highlighted.

6.6. Conclusions

In this chapter, an Improved Vulnerability Index method is developed for the construction typology of the historical centres located in Central Italy. Specific parameters are evaluated to take into account the out of plane behaviour of masonry buildings in aggregate. The results show that the Improved Method can better describe the damage scenario of the case study of Campi Alto, than another already established index-based vulnerability assessment methods, such as the Vicente (2010) method. Moreover, to further reduce the relative error between the theoretical damage and the real one, a calibration procedure based on Bayesian Inference is adopted.

It should be underlined that this calibration is performed for a specific seismic intensity ($I_{EMS98} = VIII$), which means that the accuracy of the proposed method proposed is only assured for this particular intensity. The f calibration and validation of the method using observed damage data to different intensities is therefore desirable and recommended before broader application.

On the other hand, mathematical methods (Bernardini et al. 2007) can be applied with the aim of extending the achieved results through the adoption of the calibrated model. In this way, vulnerability curves for this

construction typology can be developed for different seismic intensities. It should be noticed that this process is inevitably affected by uncertainties related to the theoretical estimation of the damage scenarios.

However, the results of the answers obtained from several experts in this field show that the level of the importance assigned mathematically to the weights is generally correct, which can be seen as a sound proof of the robustness of the method. To improve the calibration of the weights for different seismic intensities, it is possible to use the same calibration process with a bigger sample space, based on post-earthquake damage data for the same construction typology, hit by different earthquakes.

Finally, the damage scenarios evaluated with the information quality (IQ) provide better results for the forecast of damage scenarios. In this sense, a range of vulnerability values, rather than a discrete value, can be a reasonable way to account for uncertainties in the assessment, related to buildings' typologies, structural and architectural features and, besides, to the specific expertise and perception of the technicians performing the evaluation.

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Chapter 7. Definition of a quick seismic risk classification for masonry aggregates (MARS procedure)

The aim of this chapter is to provide a procedure for the quick seismic risk classification of masonry aggregates on territorial scale. The Improved method and the IQ Index are taken as reference for the evaluation of the vulnerability index of the buildings. Statistical analyses and probabilistic formulations are adopted for the definition of economic and human losses occurred in a historical centre after a seismic event. The case study of Campi Alto di Norcia is used as benchmark for the calibration of the procedure.

Chapter outline

7.1 Introduction

7.2 The Loss assessment of the case study of Campi Alto di Norcia

7.2.1 Building stock physical damage estimation scenarios

7.2.2. Evaluation of the seismic losses

7.2.3. Comparison with the real losses detected after the 2016 earthquake

7.3 Definition of the MARS (Masonry Aggregate Seismic Risk) procedure

7.4 Application of the MARS procedure in Campi Alto

7.5 Conclusions

7.6 References

7.1. Introduction

Following the last seismic events occurred in Italy, several documents analyse the number of casualties and the amount of economic resources spent on the management of emergency activities and the reconstruction of towns and historic centres (Bellicini et al. 2012), (Centro Studi Consiglio Nazionale Ingegneri 2014). The results prove that the mitigation of the seismic risk is achieved with *prevention policies*: i.e. raise awareness of seismic risk among the population, introduction of new legislative instruments for the conservation and development of the territory and use of modern tools, such as Big Data analytics and Artificial Intelligence (AI) techniques, for the evaluation of seismic risk (Atanasiu et al. 2008), (Goel et al. 2017), (Asencio-Cortés et al. 2018). The seismic risk is defined by the mathematical convolution between hazard (H), vulnerability (V) and exposure (E) (Coburn and Spence 2002), (Barbat et al. 2010), (Athmani et al. 2018):

$$R = f(H; V; E) \quad [7.1]$$

The hazard is the probability that an event occurs with a certain intensity at a specific geographical location (Klügel 2008). The exposure reflects the value, economical and human, of the elements at risk and, therefore, is related to the intended use of the structure (Mouroux and Le Brun 2006). Finally, the vulnerability describes

the capacity of the analysed building to survive a certain damage level for a seismic event of defined intensity (Calvi et al. 2006).

While the hazard and exposure are variables independent from the structure, the vulnerability requires the knowledge of the construction behaviour during an earthquake. It is clear, that the resources necessary for the study of structural performance are proportional to the importance and complexity of the building. In this sense, the masonry buildings – recognized as ‘Structural Units’ (SU) – constituting the urban environment of the historical centres are difficult to analysis, since are usually organized within Structural Aggregates (SAs), not following a well-scheduled development. With the aim of developing prevention policies for the seismic risk mitigation, it is necessary to define new tools to provide an initial screening of constructions’ fragilities on territorial scale, determining a sort of priority list for the execution of deeper investigations and for the following planning of retrofit interventions (Kassem et al. 2020).

Basing on what herein stated, the aim of the present chapter is to describe a procedure, identified with the acronym “MARS” (Masonry Aggregate Risk Scenario), for the seismic risk class estimation on urban scale of masonry buildings in aggregate, to map the most critical areas of a historical centre and to concentrate there the main resources. The *MARS procedure* is an evolution of the Improved Method described in the previous chapter, in which the seismic risk classification of existing buildings is defined with the *Vulnerability Index* (I_v) and the seismic hazard of the area, to take into account economic losses on territorial scale.

The methodology adopted to evaluate the losses in the case study of Campi Alto di Norcia is taken as reference to develop the *MARS procedure*. Starting from the application of the Improved Method in Campi Alto, different vulnerability curves are defined within a range of probable values through the information quality (IQ) index. According to these ranges, physical damage scenarios of the analysed construction typology are estimated, using a probabilistic approach. Economic losses are then evaluated (Dolce et al. 2000), based on the fragility curves developed for specific damage levels, following the EMS-98 scale. Since the probable damage scenarios are assessed between two boundaries condition, the losses are defined, considering two limited cases: a positive one, determined by a low seismic vulnerability of the building and a negative one, caused by the high vulnerability of the building. Once the fragility curves are defined, the seismic risk class can be evaluated, basing on the seismic hazard of the area where the SU is located. The seismic risk scale is formed by eight risk classes, with increasing risk from letters A⁺ to letter G, such as the seismic risk classification proposed by the Italian Code for existing buildings (DM n. 58 2017).

In order to generalize this approach, new fragility curves are evaluated, considering six vulnerability classes (V1 - V6). Economic losses are then evaluated and seismic risk classes are assigned, according to the probability of damage theoretically occurred for a specific seismic intensity and vulnerability class. Finally, with the aim to predict the expected seismic risk class on territorial scale, a correlation between seismic intensity and peak ground acceleration is introduced (Faenza and Michelini 2010).

The results provide a seismic risk probability matrix, in which the seismic risk class of the SU is determined based on the vulnerability class and the seismic hazard of the area where the SU is located. To calibrate this matrix, the seismic risk class is compared with the economic losses recorded in Campi Alto di Norcia after the 2016 earthquake. Figure 7.1 shows the work phases adopted for the definition of the *MARS procedure*.

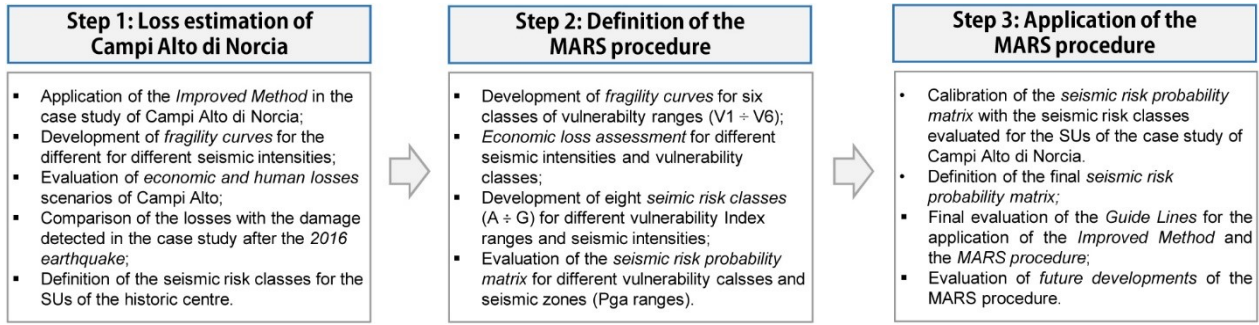


Figure 7.1: Work phases adopted for the definition of the MARS procedure

7.2. The Loss assessment of the case study of Campi Alto di Norcia

7.2.1. Building stock physical damage estimation scenarios

The application of the Improved method in the case study of Campi Alto di Norcia provides a mean vulnerability index of 45 and an average damage value of around 2.6 according to (Bernardini et al. 2007) formulation [5.3]. Besides, the information quality (IQ) index introduce two more damage scenarios through the mean I_v values for the case study analysed: 53 for the upper (+ IQ) bound and 37 for the lower one (-IQ). By changing the macroseismic intensity, it is possible to numerically develop different vulnerability curves, that represent a forecast of probable damage scenario for different earthquakes.

Once the vulnerability curves are defined different damage distributions can be assessed following a probabilistic approach. The binomial probability mass function (BDPF) allows to evaluate discrete damage distribution histograms, for different seismic intensities, as defined in Equation [7.2]:

$$p_k = \frac{n!}{k!(n-k)!} \cdot d^k \cdot (1-d)^{n-k} \text{ for } n \geq 0; 0 \leq p \leq 1 \quad [7.2]$$

Basing on the average vulnerability index evaluated for the historic centre, Figure 7.3 shows the theoretical damage distribution within different damage grades, according to the EMS-98 scale (De Matteis and Zizi 2019). The observed damage distributions can be compared with the theoretical one, considering the seismic intensity that hit the case study, that is between $I_{\text{EMS-98}} = 7.5$ and $I_{\text{EMS-98}} = 8$.

Figure 7.3a shows the expected and observed DPMs referred to global damage index of the 67 buildings of the case study, considering the seismic intensity of $I_{\text{EMS-98}} = 7.5 \approx 8$. The binomial probability mass function of the real damage overlaps the one of the expected damage scenario, suggesting that the method is robust and, therefore, a good basis for analysing other seismic intensities. Through the IQ index it is possible to assign an expected damage class for each building considering two boundary conditions (Figure 7.3b). In this sense, the upper ($D_k + IQ$) and lower bound ($D_k - IQ$) of the theoretical damage allow to identify the probable real damage classes observed in the case study.

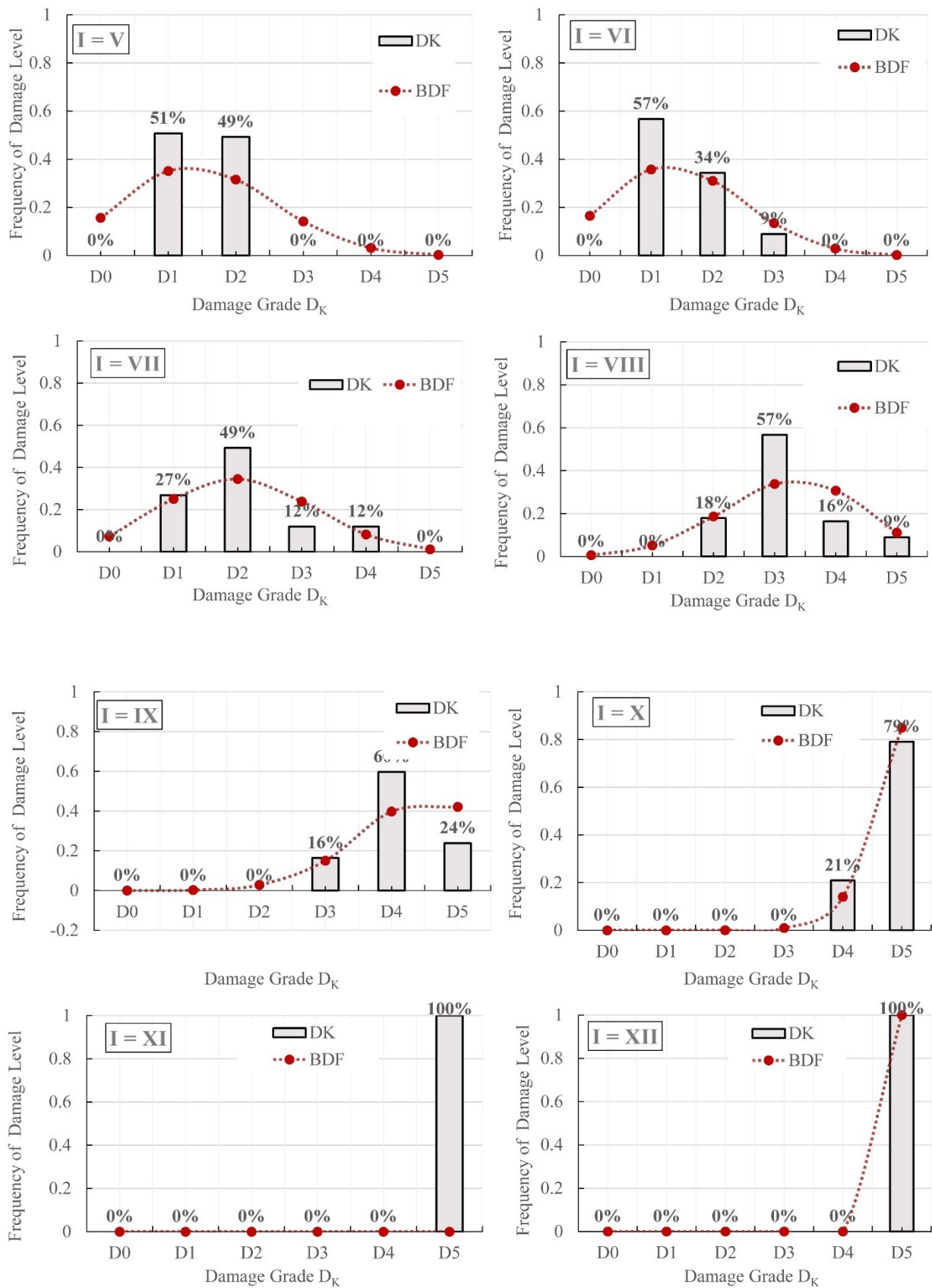
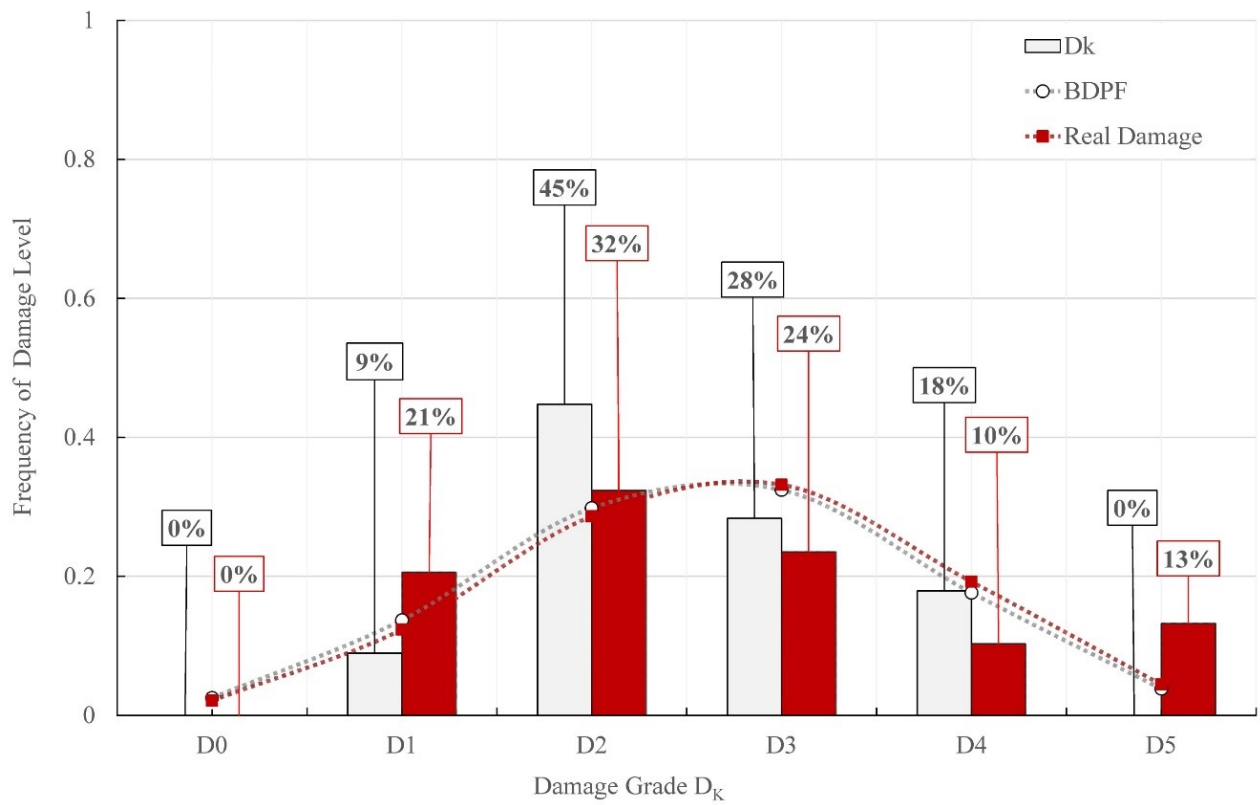
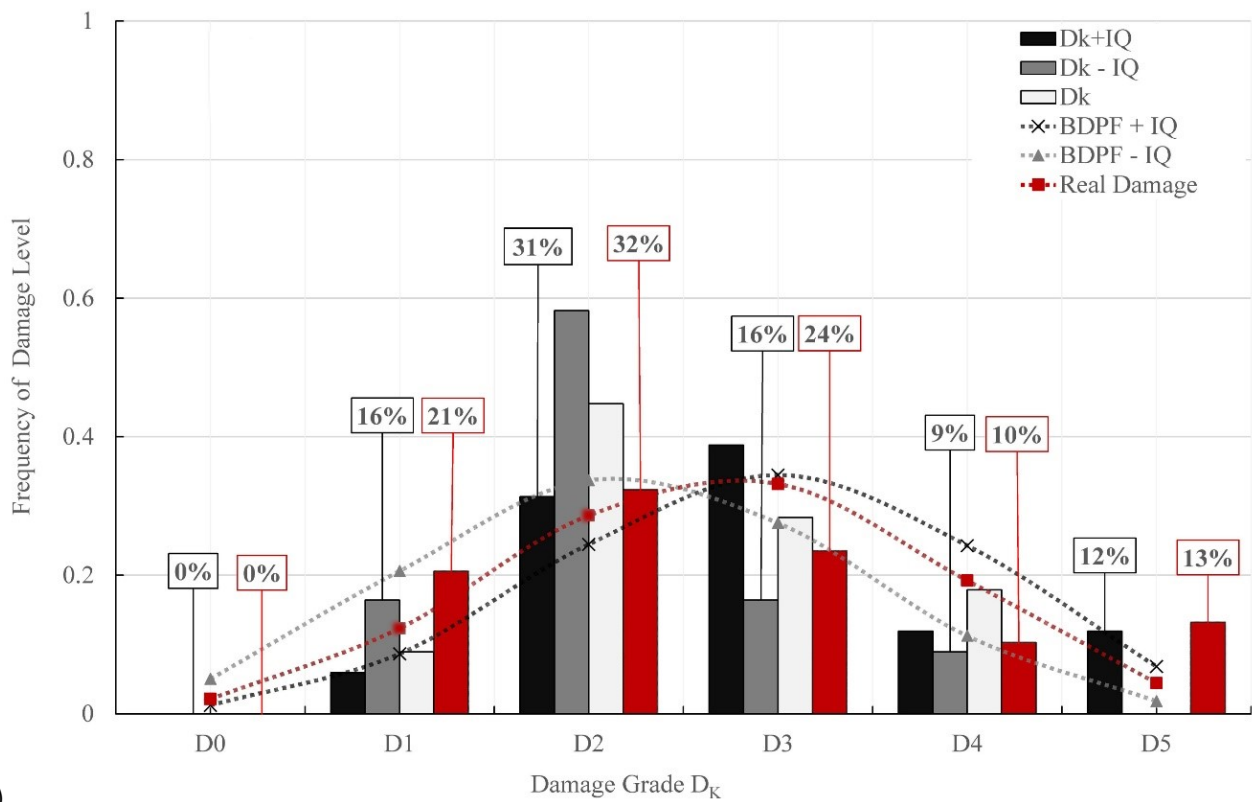


Figure 7.2: Damage distribution within the damage grade for different seismic intensities, considering the average vulnerability of the case study of Campi Alto.



a)



b)

Figure 7.3: Discrete damage distribution histograms and BDPF for $lv = 45$ (a) and for $lv = 53 (+IQ)$ and $37 (-IQ)$ (b), considering the seismic intensity of $I_{EMS-98} = 7.5 \approx 8$.

The results show that the probabilistic approach adopted can describe the damage distribution within the damage grades. Moreover, the IQ Index allows to define the boundary conditions of the problem. Fragility curves are then developed through a continuous beta probability density function, expressed by the formulation [7.3]:

$$p_B = \frac{\Gamma(t)}{\Gamma(t)\Gamma(t-r)} \cdot (x-a)^{r-1} \cdot (b-x)^{t-r-1} \text{ for } a \leq x \leq b; a=0 \text{ and } b=5 \quad [7.3]$$

While the variable “a” and “b” describe the range of the distribution, the parameter “t” and “r” define its geometry. According to (Giovinazzi 2005), (Lagomarsino and Giovinazzi 2006) and (Vicente et al. 2011) a unique value equal to 8 is assigned to the parameter t, while for the variable r the following expression [7.4] is used:

$$r = 8 \cdot \frac{\mu_D}{5} \quad [7.4]$$

Figure 7. 4 shows the different fragility curves for the historical centre of Campi Alto di Norcia. Each curve is evaluated taking into account the average seismic vulnerability of the case study ($I_v = 45.4$), defined with the calibrated Improved Method. According to the MARS procedure, two dashed curves are then added to each damage level, considering the average vulnerability index, assessed with the IQ Index ($I_v = 53.2$ for $+IQ$ and $I_v = 37.2$ for $-IQ$). The probability ($P[D > D_k]$) that the expected damage exceeds a fixed damage level for different seismic intensities is defined by an area, rather than a single value.

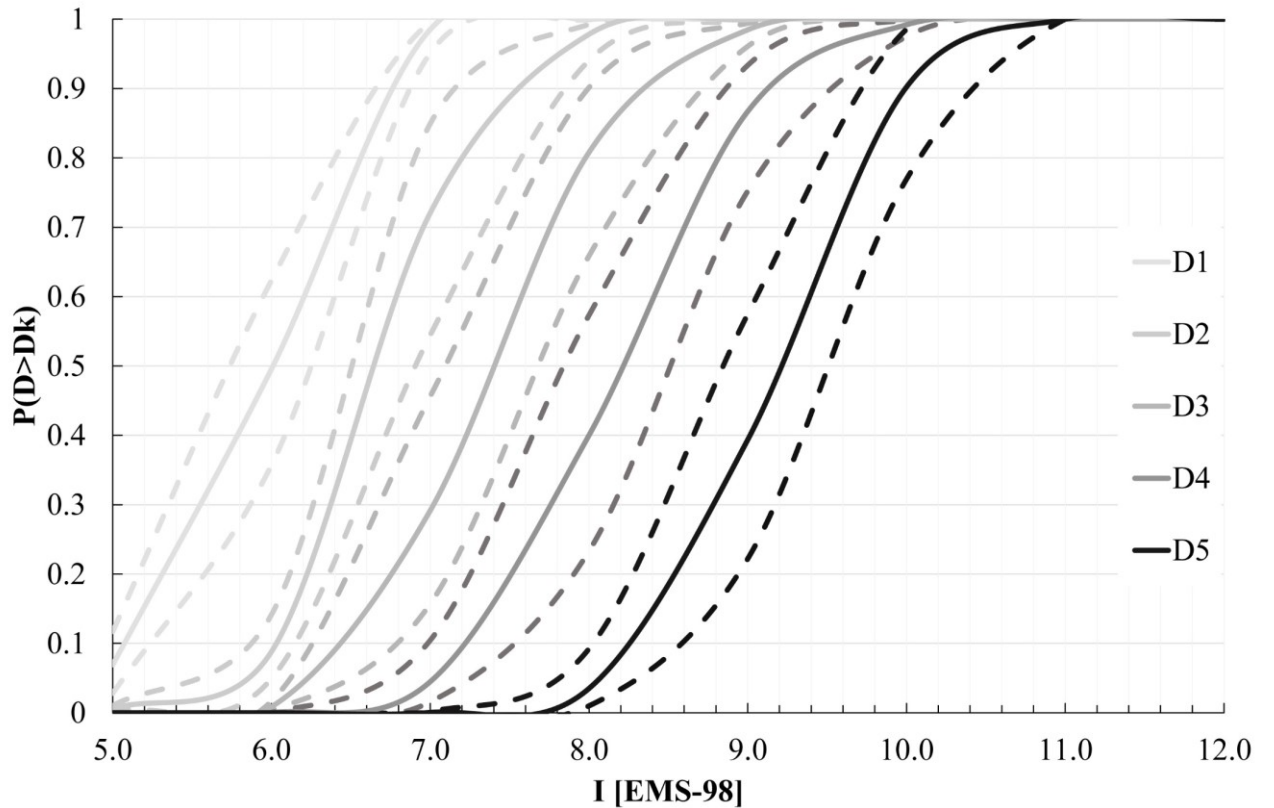


Figure 7. 4: Fragility curves of the building stock in Campi Alto di Norcia for different seismic vulnerabilities scenarios, considering the IQ Index.

7.2.2. Evaluation of the seismic losses

On the basis of the fragility curves, the expected losses are assessed, using methods available in literature and applied in other case studies (Vicente et al. 2011), (Ferreira et al. 2017), (Ortega 2018).

The correlation of the probability of occurrence of a certain damage level with the probability of building collapse defines the economic and human losses. The correlation is performed through a factor, which defines in a range from 0 to 1 the percentage of damage degree involved in the loss assessment. The value of the factors is evaluated from the damage observed after several earthquake and, therefore, change in relation to the proposal taken into account: the formulations proposed by HAZUS MH (1999) and Bramerini et al. (1995) are the most common.

The latter suggests for the analysis of collapsed and unusable buildings the following equations [7.5] and [7.6]:

$$P_{collapse} = P(D_5) \quad [7.5]$$

$$P_{unusable\ buildings} = P(D_3) \cdot W_{ub,3} + P(D_4) \cdot W_{ub,4} + P(D_5) \cdot W_{ub,5} \quad [7.6]$$

Where $P(D_3)$, $P(D_4)$ and $P(D_5)$ are the probability of occurrence for damage D3, D4 and D5, whereas $W_{ub,3}$, $W_{ub,4}$ and $W_{ub,5}$ are the weight factors for this probability function and provide the percentage of uninhabitable dwellings for each damage grade. For the loss analysis of Campi Alto di Norcia the following weights are applied: $W_{ub,3} = 0.4$; $W_{ub,4} = 0.6$ and $W_{ub,5} = 0$.

Bramerini et al. (1995) suggests for the casualties (dead and severely injured) and homelessness the formulations [7.7] and [7.8]:

$$P_{dead\ and\ severely\ injured} = P(D_5) \cdot 0.3 \quad [7.7]$$

$$P_{homeless} = P(D_3) \cdot W_{ub,3} + P(D_4) \cdot W_{ub,4} + P(D_5) \cdot W_{ub,5} \quad [7.8]$$

The rate of dead and severely injured is estimated around 30% and refers only to collapse buildings; the percentage of homeless are evaluated, considering the weight factors equal to $W_{ub,3} = 0.4$; $W_{ub,4} = 0.6$ and $W_{ub,5} = 0.7$.

Table 7.1 summarizes the probability of occurrence of different damage levels, considering the seismic vulnerability assessment performed in Campi Alto ($I_v = 45$) and the formulation previously presented.

Table 7.1: Probability of occurrence of the different damage levels considering the average vulnerability index of 45

Seismic Intensity [EMS – 98]	V	VI	VII	VIII	IX	X	XI	XII
Probability of Collapse	0%	0%	0%	3%	39%	90%	100%	100%
Probability of Unusability	0%	0%	14%	53%	52%	9%	0%	0%
Probability of Death and Severely Injured	0%	0%	0%	1%	12%	27%	30%	30%
Probability of Homeless	0%	0%	14%	55%	80%	72%	70%	70%

Figure 7.5 shows the forecast of building collapse, unusable building, homeless and casualties for the historical centre of Campi Alto di Norcia. The probability of losses assessed with the average vulnerability value of the case study, is bounded by two additional dashed curves, that define the two limit scenarios evaluated with the IQ Index, according to the Improved Method,

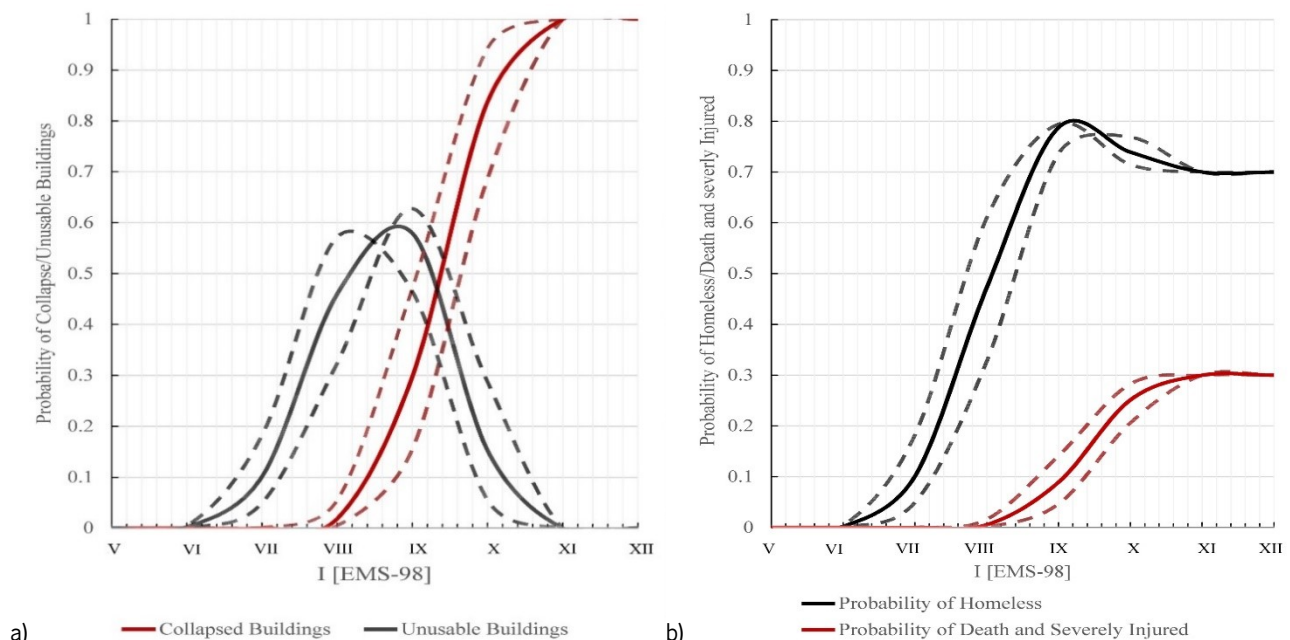


Figure 7.5: Estimation of collapsed and unusable buildings (a) and assessment of homeless and casualty rate (b) for different seismic intensities, considering the case study of Campi Alto di Norcia.

7.2.3. Comparison with the real losses detected after the 2016 earthquake

To compare the real losses detected in Campi Alto di Norcia after the 2016 earthquake with the real one, the number of collapsed and unusable buildings, as well as casualties and homeless is analysed. Demographic data (ISTAT 2001) are used to define the residents of the historical centre and information coming from the damage survey, described in chapter 4, is used as reference. While the condition of the buildings stock of a historical centre is a fixed variable during a vulnerability assessment, the probability of casualties, as well as homeless, is hard to define through a single rate. The presence of people inside the buildings during an earthquake depends, indeed, on many variables, such as the intensity and time period of the earthquake swarm, as well as the time of year and day, when the mainshock takes place. In the same way, the probability of homeless is influenced by the intended use of the building (second home, business, ...) and by the environment condition of the

historical centre after the seismic event (accessible streets, facilities condition, ...). In this sense, the expressions [7.7] and [7.8] are a first rough estimation about the expected human casualties and homeless for a specific seismic intensity and, therefore, must be improved through the cross correlation of other data (Bernardini et al. 2010), (Bernardini et al. 2016) (Aguado et al. 2018) (Giuliani et al. 2019).

Table 7. 2 shows the expected number of collapsed and unusable buildings for different seismic intensities. The real number of unusable buildings, after the 2016 seismic event, is equal to the assessment performed for the seismic intensity between $I_{EMS-98} = VII$ and $I_{EMS-98} = VIII$, taking into account the average vulnerability of 45. On the contrary, if it is considered the increased vulnerability through the IQ index ($I_v + IQ = 53$), the number of expected collapsed buildings is more similar to the real one

Table 7. 2: Estimation of the collapsed and unusable buildings for different seismic intensities, where in red the number of collapsed and unusable buildings after the 2016 seismic event are highlighted.

Economic Losses (Total Number of Buildings: 67)		Seismic Intensity, I (EMS-98)								
		V	VI	VII	2016 Seismic Event	VIII	IX	X	XI	XII
$I_v = 37.2$	Collapsed Buildings	0	0	0	7	1	15	52	67	67
	Unusable Buildings	0	0	5	35	26	41	14	0	0
$I_v = 45.4$	Collapsed Buildings	0	0	0	7	2	26	60	67	67
	Unusable Buildings	0	0	10	35	35	35	6	0	0
$I_v = 53.1$	Collapsed Buildings	0	0	0	7	6	39	66	67	67
	Unusable Buildings	0	0	16	35	41	26	1	0	0

Table 7.3 summarizes the expected number of homeless, dead and severely injured for different seismic intensities, compared to the real number of casualties and homeless of the case study analysed after the earthquake of 2016. As previously said, this estimation is influenced by different variables that aren't taken into account. The seismic event that hit the case study analysed is defined by different main shocks (24.08.2016 – $M_w = 5.4$; 30.10.2016 – $M_w = 6.5$), where the earthquake swarm lasted for almost 6 months. In this sense, during that period many residents left the area and no casualties occurred. Since the internal system of roads of Campi Alto di Norcia was damaged by the earthquake, all the buildings of the historical centre became inaccessible and all the residents homeless.

Table 7.3: Estimation of homeless, dead and severely injured for different seismic intensities, where in red the number of homeless, dead and severely injured after the 2016 seismic event are highlighted.

Human Losses (Population: 172 Residents – ISTAT 2001)		Seismic Intensity, I (EMS-98)								
		V	VI	VII	2016 Seismic Event	VIII	IX	X	XI	XII
$I_v = 37.2$	Death and Severely Injured	0	0	0	0	1	11	40	52	52
	Homeless	0	0	12	172	69	132	130	120	120
$I_v = 45.4$	Death and Severely Injured	0	0	0	0	2	20	47	52	52
	Homeless	0	1	25	172	95	137	125	120	120
$I_v = 53.1$	Death and Severely Injured	0	0	0	0	5	30	51	52	52
	Homeless	0	1	42	172	116	135	121	120	120

Through the GIS environment it is possible to compare the real damage level detected in the case study of Campi Alto with the probability of occurrence of different economic and human losses, considering the seismic intensity that hit the case study.

While Figure 7.6 shows the probability of occurrence of economic losses, in terms of collapsed and unusable buildings, Figure 7.7 defines the probability of death/severely injured and homeless within the case study. The results show that the statistical formulations [7.3] and [7.4] applied, are useful to describe the probability of occurrence of a certain damage level, according to the EMS – 98, for specific seismic intensities. It is also note, that the human losses are difficult to forecast, especially for a simplified method on territorial scale, which cannot take into account too many different factors.

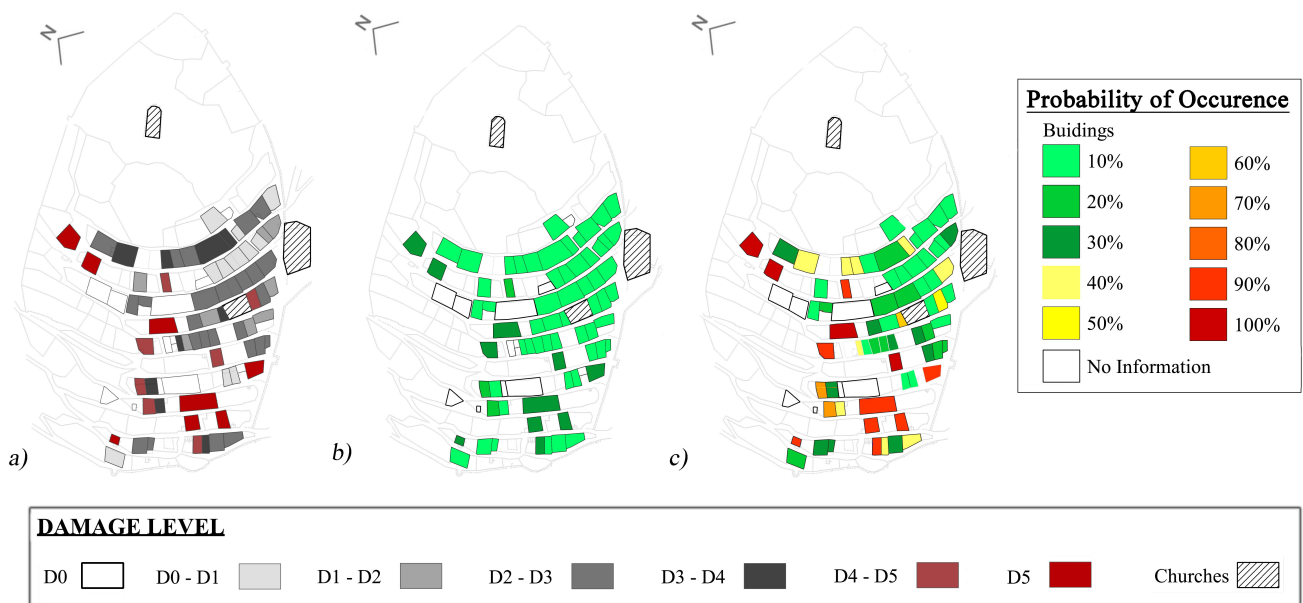


Figure 7.6: Comparison between the real damage detected (a), the probability of occurrence of collapsed (b) and unusable buildings (c).

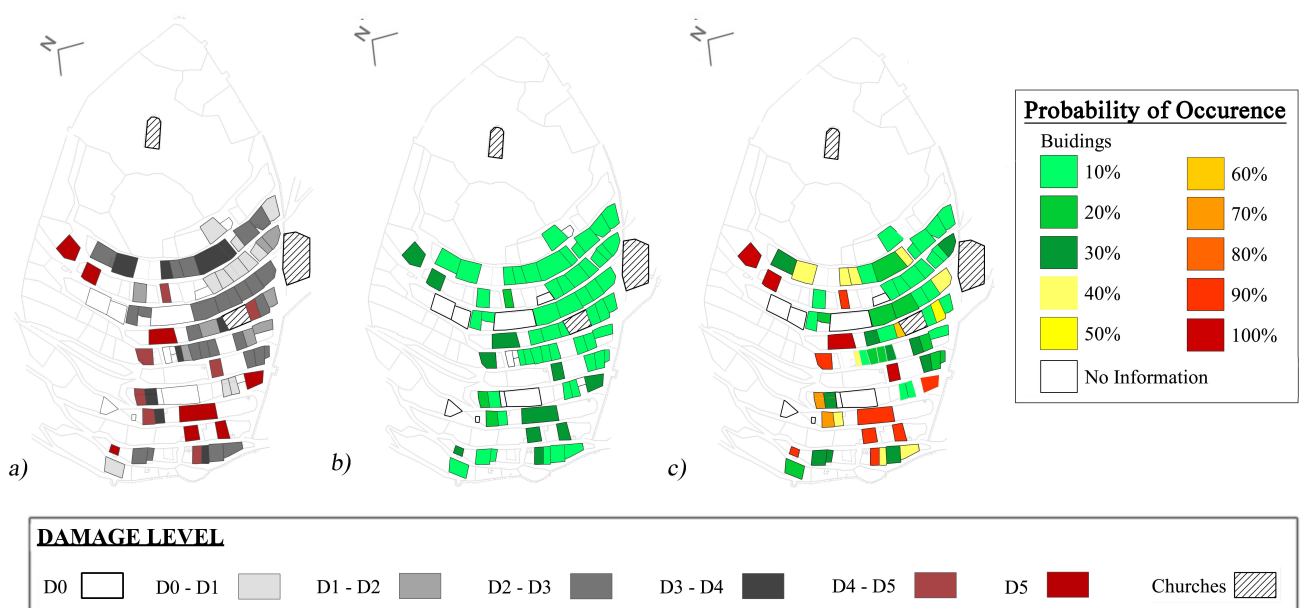


Figure 7.7: Comparison between the real damage detected (a), the probability of occurrence of death/severely Injured (b) and homeless (c).

7.3. Definition of the MARS (Masonry Aggregate Seismic Risk) procedure

The comparison between the losses estimation and the real damage detected in Campi Alto after the 2016 earthquake, shows that a simplified method for risk assessment is reliable for economic, rather than human, losses. Since the purpose of the MARS procedure is to evaluate the seismic risk class of a masonry building in historic centre on a territorial scale, the estimated damage is economically interpreted, as the ratio between the repair cost and the replacement cost (building value).

Several formulations are developed in literature, to correlate the EMS – 98 damage degrees with an economic damage index (D_k), based on analyses performed with post-earthquake damage data. Table 7.4 shows the formulations proposed by (Bramerini et al. 1995), (Rojahn et al. 1985) and (Dolce et al. 2006).

Table 7.4: Correlation between damage level and damage index

Damage index (D_k)	0	1	2	3	4	5
(Bramerini et al. 1995)	0.000	0.010	0.100	0.350	0.750	1.000
(Rojahn et al. 1985)	0.000	0.050	0.200	0.550	0.900	1.000
(Dolce et al. 2006)	0.005	0.035	0.145	0.305	0.800	0.950

The expression of damage index proposed by Dolce et al. (2006) is adopted for the MARS procedure. The results obtained by these authors are evaluated by the statistical analysis of the data collected with the GNDT-SSN procedure, after the Umbria-Marche (1997) and Pollino (1998) earthquakes (Vicente et al. 2011) and, therefore, are based on the damage occurred to the same construction typology with which the Improved Method is developed. Moreover, the order of magnitude of damage index proposed by Dolce et al. (2006) agree with the damage scenario of Campi Alto; the investigation carried out by Di Pasquale and Goretti (2001) also confirms that the damage index assumes a similar value of the damage index for damage grade 4 and 5 and a greater difference between the damage index for the lower damage grades of 1 and 2.

Basing on what herein stated, the conditional probability of loss $[L]$ for different seismic intensities $[I]$, defines the seismic risk class of the building. The latter is assessed, according to the formulation [7.9], as the multiplication of the conditional probability of the loss for each damage level $P[L|D_k]$ and the conditional probability of the damage level evaluated for different vulnerability indexes and seismic intensities $P[D_k|I_V, I]$.

$$P[L|I] = \sum_{k=1}^5 \sum_{I_V=0}^{100} P[L|D_k] \times P[D_k|I_V, I] \quad [7.9]$$

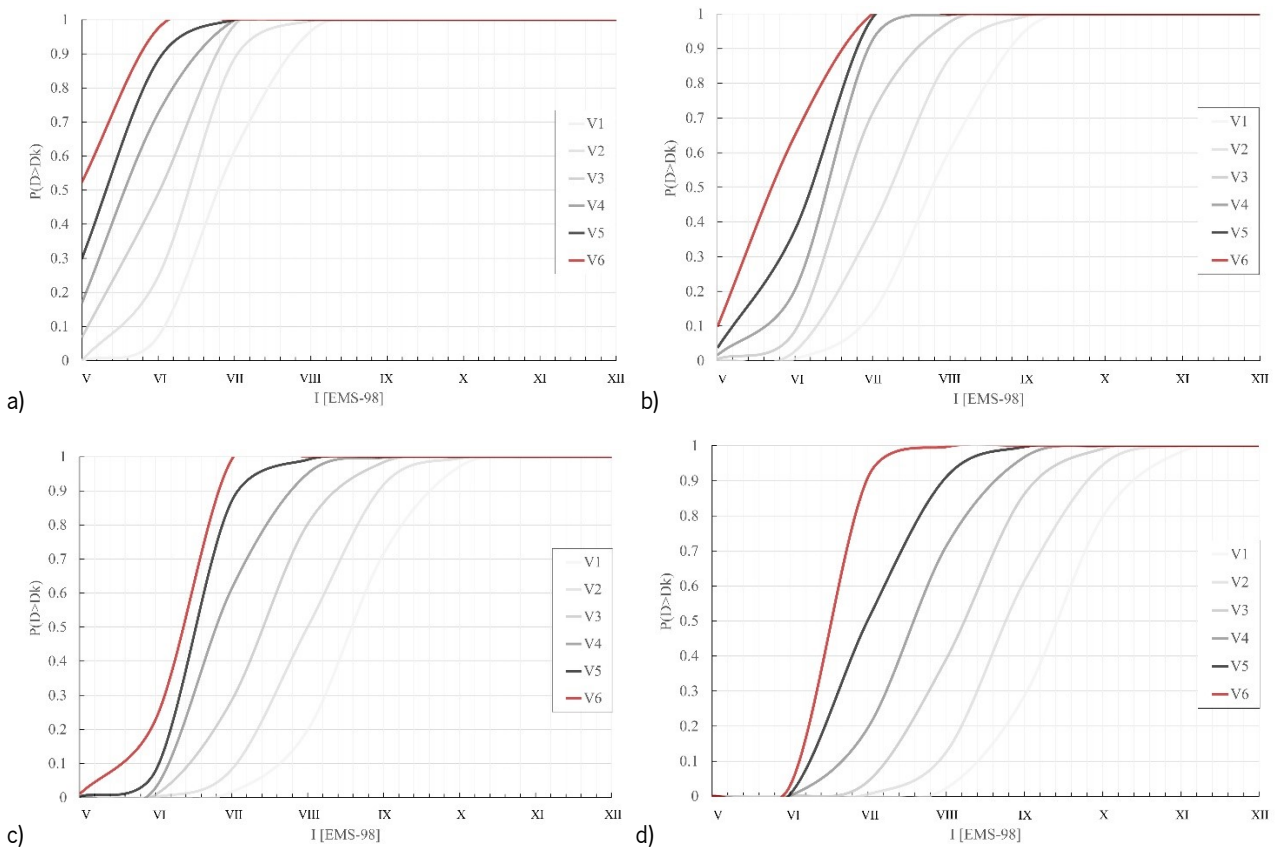
In order to define the probable damage for different vulnerability indexes and seismic intensities, six vulnerability classes with increasing vulnerability from V1 to V6 are proposed (Table 7.5), according to the distinction made by the European Macroseismic Scale EMS-98 (Grünthal 1998). Each vulnerability class express a Vulnerability Index range, characterized by a mean value and two boundary conditions. The value of the ranges is evaluated

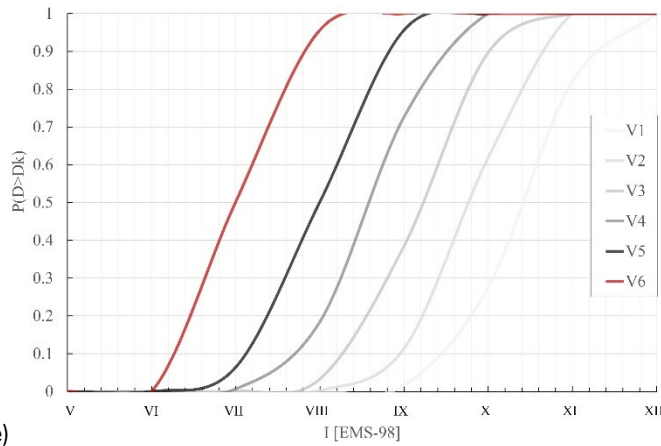
through the damage surveyed in the case study, in which the assessments, performed with the Improved method, show in average the relation between vulnerability index and damage class detected.

Table 7.5: Identification of the vulnerability classes according to the vulnerability Index variation

Vulnerability Class	Vulnerability Index
V1	$0 \leq I_v < 15$
V2	$15 \leq I_v < 30$
V3	$30 \leq I_v < 45$
V4	$45 \leq I_v < 60$
V5	$60 \leq I_v < 75$
V6	$75 \leq I_v < 100$

Based on the six vulnerability classes, fragility curves are developed for different seismic intensities, considering the continuous beta probability density function [7.3] and the formulation [7.4] for the assessment of the expected damage. Figure 7. 4 shows the fragility curves for the six vulnerability classes, as defined in Table 7.5, and for different damage levels. According to the expression [7.9], the probability of loss [L] is evaluated by multiplying the damage index with the probability of occurrence of different damage levels (D1 - D5) for each vulnerability index [V] and seismic intensity [I]. The seismic risk classes are, finally, introduced to describe the conditional probability of loss: eight classes are identified, with increasing probability of loss from letters A+ to letter G. Figure 7.9 and Table 7.6 show the graphical representation of formulation [7.9] according to the six vulnerability classes.





e)

Figure 7.8: Fragility curves of the building stock in Campi Alto di Norcia considering six vulnerability classes and the damage levels D1 (a), D2 (b), D3 (c), D4 (d), D5 (e), according to the EMS-98 scale.

Table 7.6: Identification of the seismic risk class according to the economic loss

Seismic Risk Class	Percentage of loss
A+	0.00 - 12.5%
A	12.6 - 25.0%
B	25.1 - 37.5%
C	37.6 - 50.0%
D	50.1 - 62.5%
E	62.6 - 75.0%
F	75.1 - 87.5%
G	87.6 - 100%

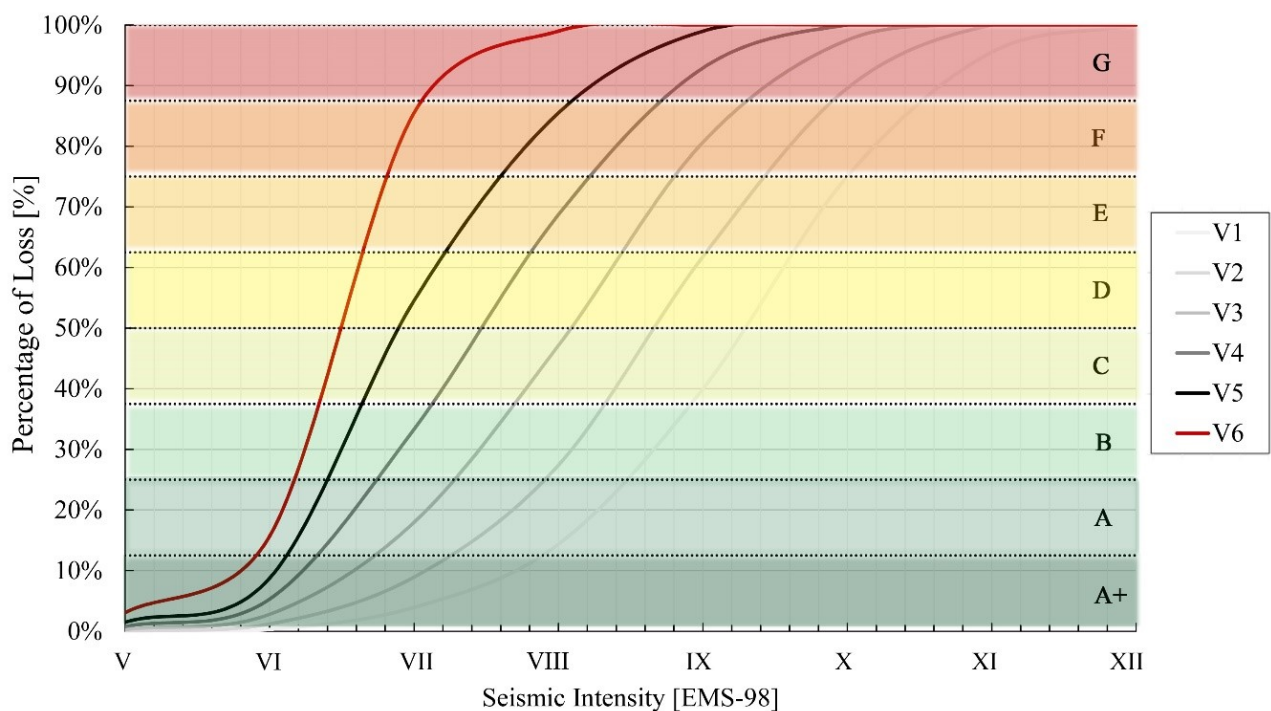


Figure 7.9: Graphic representation of the seismic risk class in relation to the vulnerability classes and seismic intensities.

Basing on Figure 7.9, a probability matrix for the seismic risk class assessment is defined for different intensities (Table 7.7). The latter measures the probability of losses of a SU in aggregate, once the Vulnerability Index and the seismic intensity are evaluated.

Table 7.7 Seismic risk class matrix in relation to the vulnerability classes and the seismic intensities. An example of how the probability matrix works for the seismic risk class assessment is shown in red.

Vulnerability Class	Seismic Intensity (EMS -98)							
	V	VI	VII	VIII	IX	X	XI	XII
V1	A+	A+	A+	A	C	F	G	G
V2	A+	A+	A+	B	D	G	G	G
V3	A+	A+	A	C	F	G	G	G
V4	A+	A+	B	E	G	G	G	G
V5	A+	A+	D	F	G	G	G	G
V6	A+	A	F	G	G	G	G	G

Since the seismic intensity is based on the observed damage after a seismic event, a correlation between the seismic intensity and the pga is introduced, in order to link the seismic hazard with the area where the SU is located. In this sense, the seismic risk class can be predicted rather than measured.

It is note, that many logarithmic correlations between seismic intensities and PGA are present in the scientific literature, based on different earthquake data (Guagenti and Petrini 1989), (Margottini et al. 1992), (Wald et al. 1999), (Faccioli et al. 2006), (Faenza and Michelini 2010). Since these correlations are influenced by the spectral response of the considered seismic input (such as peak number, acceleration frequency, subsoil typology, etc), they are earthquake dependent and, therefore, the best correlation to be adopted for prediction purposes should be the one tuned on the sample environment area or, in case it is not available, another one adjusted on similar conditions (De Matteis and Zizi 2019). In this sense, the correlation introduced in the MARS procedure is the one suggested by Faenza and Michelini (2010) [7.10]:

$$I_{MCS} = 1.68 + 2.58 \cdot \log(\text{pga}) \quad [7.10]$$

The data set used for this formulation are, indeed, assembled for earthquakes that have occurred in Italy in the time period 1972–2004 (Luzi et al. 2008). Since the Italian National Institute of Geophysics and Volcanology (INGV 2019) uses the expression [7.10] to derive the pga values from the MCS intensity maps, it can be considered robust and reliable. The formulation [5.2] is applied to correct the formulation [7.10], in order to correlate the MCS scale with the European Macroseismic Intensity Scale.

Finally, the matrix of the seismic risk class is developed, through which it is possible to evaluate the risk class of the building, according to its vulnerability class and the seismic hazard of the area, where the building is located (Table 7.8). The hazard of the area is identified through the seismic zone to which it belongs, as defined by the Italian seismic classification (D.P.R. 380 2001), in which the seismic hazard of the Italian territory is

divided into four zones, with different acceleration intervals, having 10% probability of exceeding in 50 years (Table 7.9).

Table 7.8: Seismic risk class matrix in relation to the vulnerability class and the seismic hazard for the economic losses. An example of how the probability matrix works for the seismic risk class assessment is shown in red.

Vulnerability Class	Seismic Zone			
	1	2	3	4
V1	A	A+	A+	A+
V2	B	A	A+	A+
V3	C	B	A	A+
V4	E	D	B	A+
V5	F	E	D	A+
V6	G	F	F	A

Table 7.9: Italian seismic classification according to (D.P.R. 380 2001).

Seismic Zone	Short Description	pga range
1	It is the most dangerous area. The probability of a strong earthquake occurring is high.	$\text{pga} > 0.25$
2	In this area strong earthquakes are possible.	$0.15 < \text{pga} \leq 0.25$
3	Strong earthquakes in this area are less likely than in zones 1 and 2.	$0.05 < \text{pga} \leq 0.15$
4	It is the least dangerous area: the probability of an earthquake occurring is very low	$\text{pga} \leq 0.05$

Figure 7.10 summarizes the procedure followed to define the seismic risk classes, in relation to the vulnerability index, the seismic intensity and the seismic zone.

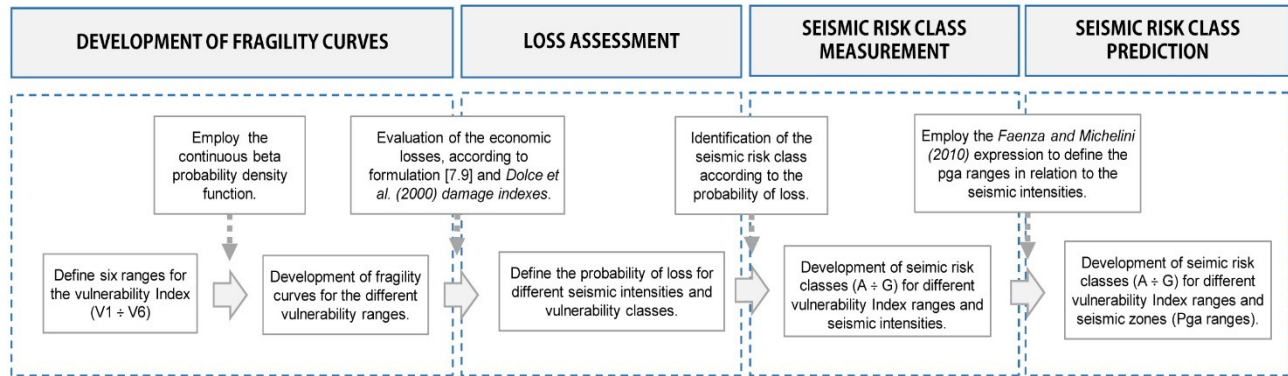


Figure 7.10: Procedure followed for the seismic risk class assessment through the MARS procedure

7.4. Application of the MARS procedure in Campi Alto

The MARS procedure is applied to the case study of Campi Alto di Norcia, in order to calibrate the seismic risk class matrix with the economic losses detected after the 2016 earthquake. In this sense, a comparison between the seismic risk classes of the SUs is performed, considering the real and the theoretical damage scenario of the case study for the seismic intensity of $I_{\text{EMS-98}} = 7.65 \approx 8$.

The seismic risk class based on the real damage scenario is assigned to the SUs of the historical centre, through the damage index suggested by Dolce et al. (2006): i.e. the damage level D1 is equal to the percentage of loss 3.5% and, therefore, to the seismic risk class A+; instead, the damage level D5 corresponds to the total collapse of the building (100% of loss) and to the seismic risk class G. In the case of intermediate damage degrees, a weighted average of the losses between the damage classes involved is performed.

The theoretical seismic risk class of the SUs is evaluated, first, with the formulation [7.3], using the vulnerability index of the buildings assessed through the Improved Method and the IQ Index. In this case, the probability of loss is defined by multiplying the damage indexes with the probability of occurrence of different damage levels, estimated with the continuous beta probability density function and the Bernardini et al. (2007) expression. Afterwards, the theoretical seismic risk class is also defined with the risk class matrix (Table 7.8) for the seismic zone 1 and the vulnerability classes of the buildings derived from the Improved Method.

The frequency of each seismic risk class is evaluated, considering the amount of buildings for each class, taking into account the real and theoretical damage scenario. The results achieved, summarized in Table 7.10, shows that the number of the real risk classes is on average within the range of expected classes, evaluated with the IQ Index. Nevertheless, the variance of the real classes with the theoretical ones is still high, not allowing to define the correct class and reducing the accuracy of the forecast.

Furthermore, the prediction of the classes carried out through the risk class matrix has provided more conservative results than the real ones for low values of vulnerability index: since Campi Alto di Norcia has a seismic hazard that correspond to the seismic zone 1, the seismic risk class A or B is foreseen for a vulnerability class of V1 or V2. On the contrary, many buildings of the case study show a better seismic capacity with very low percentage losses (A+) in several cases. The risk class matrix is, instead, reliable for high percentages of economic losses, such as seismic risk class F and G.

Table 7.10: Comparison of the real and expected seismic risk class for the SUs of the case study

Seismic Risk Class	Probability of Damage	EXPECTED RISK CLASS			RISK CLASS MATRIX	REAL RISK CLASS
		Iv - IQ	Iv	Iv + IQ	Iv	Iv
A+	0.00 - 12.5%	12	11	5	0	15
A	12.5 - 25.0%	35	18	15	2	22
B	25.0 - 37.5%	7	19	14	11	8
C	37.5 - 50.0%	3	5	15	25	6
D	50.0 - 62.5%	5	3	2	0	0
E	62.5 - 75.0%	5	3	4	16	1
F	75.0 - 87.5%	0	5	2	5	6
G	87.5 - 100%	0	3	10	9	9

Figure 7.11 and Figure 7.12 shows the seismic risk class distribution within the case study of Campi Alto di Norcia through the GIS environment.

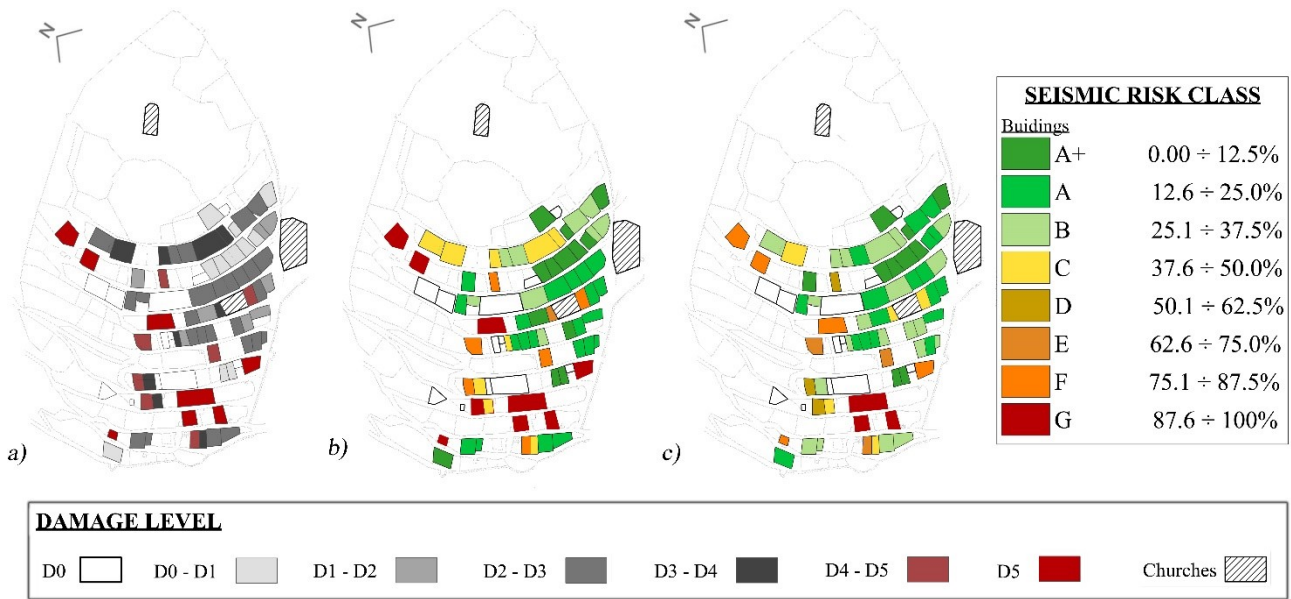


Figure 7.11: Comparison between the real damage detected in the case study (a), the real seismic risk class (b) and the expected seismic risk class considering the formulation [7.3] (c).

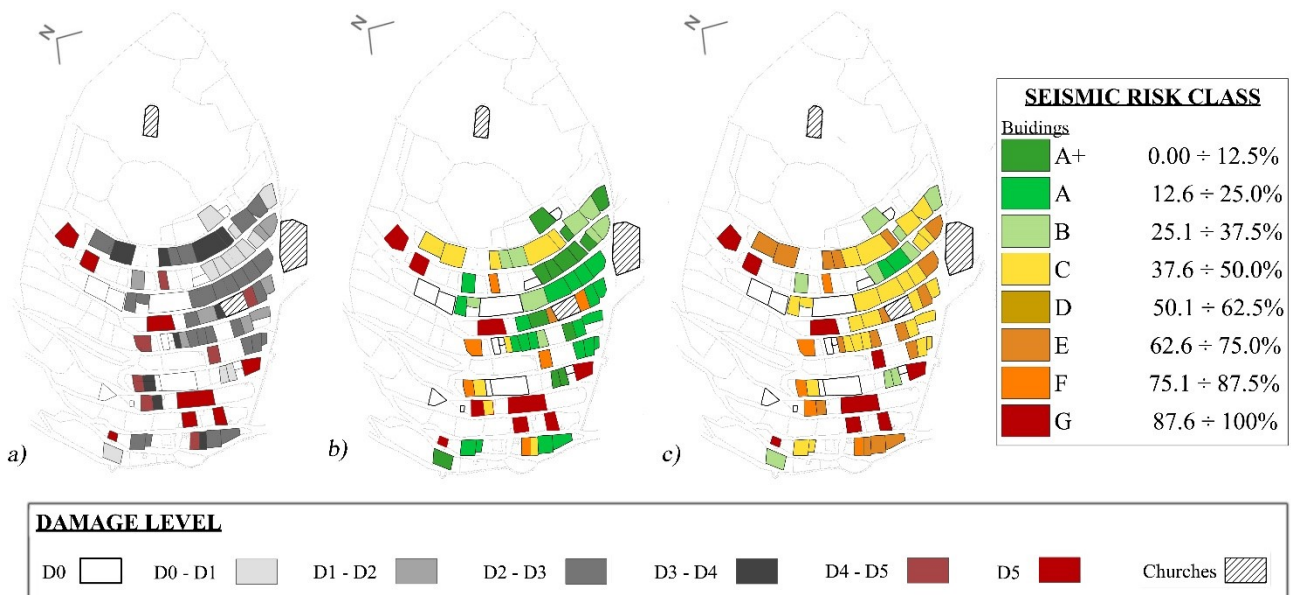


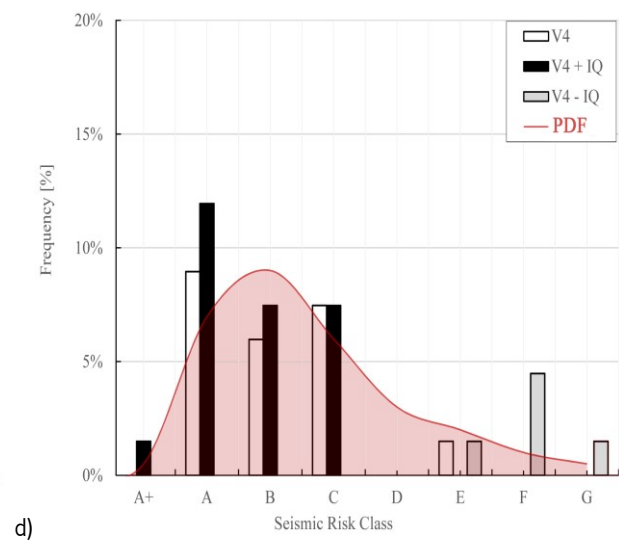
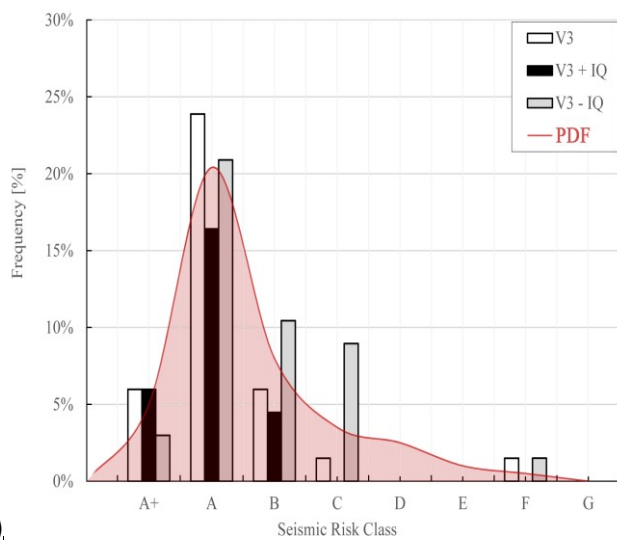
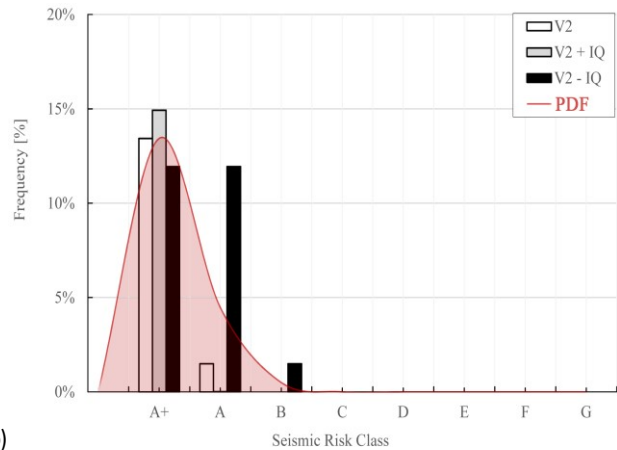
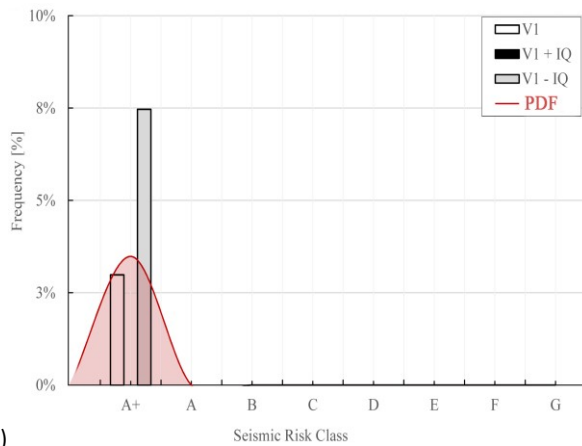
Figure 7.12: Comparison between the real damage detected in the case study (a), the real seismic risk class (b) and the forecast seismic risk class considering the risk class matrix (c).

Basing on what herein stated, the calibration of the seismic risk class matrix is performed by measuring the relative frequency of the vulnerability classes obtained for each building of the case study, through the Improved Method and the IQ Index, among the real seismic risk classes. It is therefore possible to define the probability with which the vulnerability class falls into the seismic risk class, according to the construction typology analysed and the seismic intensity taken into account $I_{EMS-98} = 7.65 \approx 8$. The statistical analyses of the data allow to calibrate the seismic risk class: the mode of the probability density function (PDF) defines the prevailing seismic risk class, while the Average and Standard Deviation (SD), shows the seismic risk classes involved for each vulnerability class.

Figure 7.13 and Table 7.11 shows the distribution of the vulnerability classes within the seismic risk classes. In terms of histograms and PDF.

Table 7.11: Distribution of the vulnerability classes within the seismic risk classes.

Parameter	V1	V2	V3	V4	V5	V6
Average	0.03	0.07	0.22	0.37	0.83	0.96
SD	0.00	0.06	0.12	0.22	0.21	0.08
Average + 2*SD	0.04	0.19	0.46	0.82	1.00	1.00
Average - 2*SD	0.00	0.00	0.00	0.00	0.00	0.00
MAX	0.04	0.25	0.87	0.97	1.00	1.00
MIN	0.02	0.02	0.02	0.09	0.20	0.80
Mode	0.03	0.04	0.20	0.27	0.81	0.92



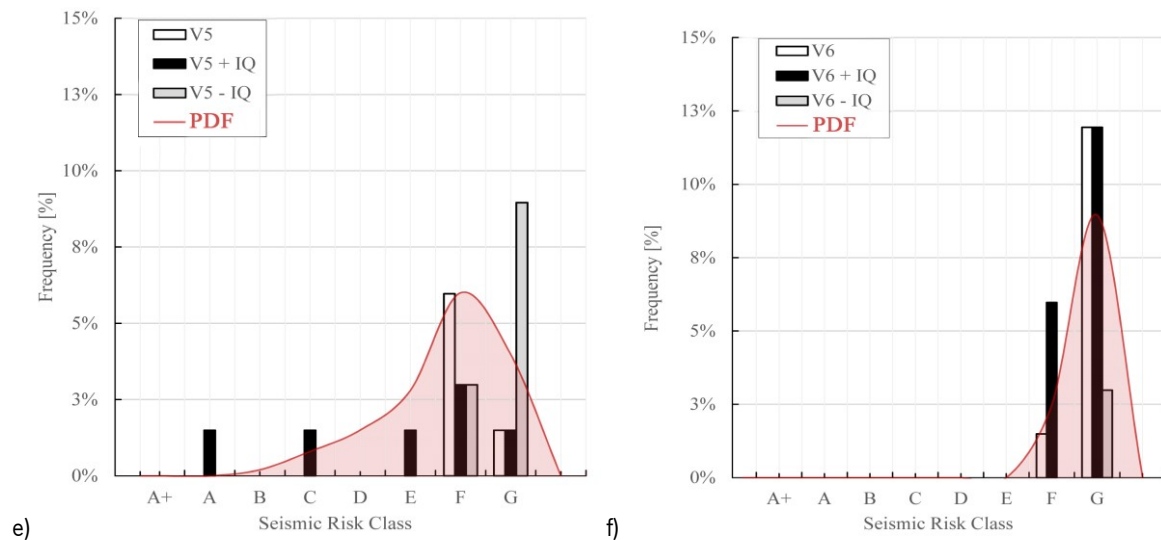


Figure 7.13: Frequency of the seismic risk classes within the case study, considering the vulnerability classes V1 (a), V2 (b), V3 (c), V4 (d), V5 (e) and V6 (f).

The matrix for the seismic risk class in the MARS procedure is redefined according to the results achieved: a probabilistic approach determines the relationship between seismic risk and vulnerability class. Table 7.12 shows the new seismic risk matrix for the seismic zone 1, in which the most probable seismic risk class is highlighted by a "circle", while the "solid line" and the "dashed line" consider the probable and the less likely or exceptional seismic risk class. It is note, that only the vulnerability classes of the seismic zone 1 are calibrated, since Campi Alto has a high seismic hazard. The seismic risk matrix remains, therefore, unchanged for the other seismic zones (Table 7.13).

Table 7.12: Seismic risk class matrix calibrated for the seismic zone 1.

Zone 1	A+	A	B	C	D	E	F	G
V1	○							
V2	○	—	—					
V3		○	—	—				
V4			○	—	—			
V5						—	○	—
V6								○

Table 7.13: Seismic risk class matrix for all the seismic zones, where the calibrated classes are highlighted in red.

Vulnerability Class	Seismic Zone			
	1	2	3	4
V1	A+	A+	A+	A+
V2	A+	A	A+	A+
V3	A	B	A	A+
V4	B	D	B	A+
V5	F	E	D	A+
V6	G	F	F	A

7.5. Conclusions

The MARS procedure proposes a quick technique for the preliminary screening of seismic risk of masonry buildings in aggregates on urban scale. The results of the vulnerability assessments carried out with an Empirical/Expert base approach, such as the Improved Method, are compared with the seismic hazard of the territory in order to define the possible damage scenarios. In this sense, the MARS procedure can be considered as an evolution of the Improved Method, as it completes the vulnerability assessment of a historical centre through the expected losses.

Campi Alto di Norcia is taken as reference for the losses evaluation occurred to the construction typology analysed, considering the seismic intensity which hit central Italy in October 2016. The comparison between the theoretical probability of occurrence of economic and human losses and the real damage scenario detected in the case study allows several observations: the human losses are difficult to forecast, especially for a simplified method on territorial scale, and require the analyses of the data with different approaches. The number of casualties of a seismic event is influenced by the amount of people present in an urban centre during the earthquake, i.e. the time of day and of the year in which the earthquake occurs. Since social habits are complex non-linear systems characterized by unpredictable dynamics, it is difficult to provide reliable results. While in linear systems, a small change in the initial state causes a slight alteration in its final state, in a nonlinear system small differences of the primary conditions can produce unpredictable differences in following behaviours (Liu et al.), (Jon et al. 2016). An example is provided by the seismic events of 2016 in Central Italy: the first strong earthquakes struck the historic centres of Accumoli and Amatrice at night during the tourist season and several victims were recorded. Even if the magnitude of the seismic events that hit Campi Alto di Norcia ($M_w = 6.5$), are almost the same as Accumoli and Amatrice ($M_w = 6.0$), no casualties were recorded in the case study, as the earthquake occurred after several seismic swarms that alarmed the population. For these reasons, the MARS procedure, which is a quick and simplified methodology, focuses on forecasting economic rather than human losses. The analyses of economic losses provide more reliable results, since they are based on linear system, that is the building stock, analysed during the vulnerability assessment.

Moreover, the statistical analyses of the losses allow to define a seismic risk class matrix, the final tool of the MARS procedure. It is note, that the physical damage of the case study is caused by a seismic event with a specific intensity ($I_{EMS98} \approx 8$): this means that the seismic risk probability matrix is verified only with a seismic intensity similar to that used. The performance of forecast and prediction models of the MARS procedure can be improved by increasing the sample space of the data used for the statistical analysis, in particular to take into account additional seismic intensities (Kossobokov et al. 2015). In this way, it would be possible to take advantage of the potential proposed by the MARS procedure, for the classification of the territorial seismic risk of other historical centres with the same construction typology. However, the comparison of the expected seismic risk classes, evaluated through the matrix, with the real damage observed, suggests that its application provides conservative results and therefore defines the worst case scenario of losses.

Starting from the probability of losses, suggested by the seismic risk class of the MARS procedure, it is possible to estimate the economic value of the reconstruction costs after the seismic event. The prediction of the repair costs of seismic damage is developed on the basis of damage scenarios, which occurred in a historic centre due to a seismic event, multiplying the percentage of loss with the replacement cost (Ferreira et al. 2017). The evaluation of the latter is not simple and differs by construction typology and location of the urban centre.

Furthermore, some authors suggest that intervention costs following an earthquake event can be affected by the cost increase of materials, equipment and workmanship as a result of the rise in demand (Olsen 2012).

The report developed by ReLUIS (2010) summarizes the reconstruction costs of the municipalities affected by the earthquake in Abruzzo on 6 April 2009 and statistically defines the average cost of repair and reconstruction of the building typology analysed. The correlation of the results of the MARS procedure with the information coming from this document can be taken as a reference, for the evaluation of the repair cost after an earthquake for the historic centres in the Apennines of Central Italy. Finally, the theoretical estimation of damage scenarios, seismic risk classes and reconstruction costs of masonry buildings in old urban centres can be a valuable tool for the seismic risk mitigation. Indeed, by correlating the retrofit cost with the economic losses, it is possible to quantify the effect of building strengthening from an economical viewpoint and to highlight the importance of preserving traditional constructions systems compared to the cost of managing the seismic emergency and the building reconstruction following a seismic event (Maio et al. 2019) .

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Chapter 8. Conclusions and outlooks

This chapter summarizes the results achieved in the thesis: the answers to the objectives, specified in Chapter 1, are described in detail. The tool and procedure defined during the research activity are presented in a global context. Finally, the limitations of the latter are also shown, providing suggestions for their improvement.

Chapter outline

8.1 Introduction

8.2 Summary of the results achieved

8.2.1 Objective 1: Literature review and database development for the analyses of historic centres hit by seismic events.

8.2.2 Objective 2: Evaluation of the accuracy of different Iv methods in predicting the damage scenarios.

8.2.3 Objective 3: Development of a seismic vulnerability assessment method for masonry Aggregate.

8.2.4 Objective 4: Definition of a codified procedure for the evaluation of economic impact of seismic events on historical centres.

8.3 Future research

8.1. Introduction

The purpose of the present PhD research is to provide further tools for the seismic risk mitigation of historical city centres characterized by high cultural and historical value, which are not provided by adequate analysis' methods for achieving reliable large-scale vulnerability assessments. Since different research fields address these issues through many approaches and from different points of views, this study organizes and reviews the current state of the art, through an in-depth analysis of the scientific literature. Moreover, the effects of earthquakes in a historic centre are taken into account, analysing the retrofit interventions and the seismic damage observed in the case study of Campi Alto di Norcia throughout its history and after the 2016 earthquake.

The data acquisition carried out directly in the historic centre and their post-processing in a GIS environment allows the correlation of different information to better understand the causes of certain structural failures: the comparison between the damage level and different retrofits typologies carried out before and after the seismic events measures the efficiency of specific technologies.

The result of this activity is the definition of a database, from which an Improved method is developed for the seismic vulnerability assessment of historic centres on urban scale. This tool allows to define probable damage scenarios for different seismic intensities and provides the base on which statistical analyses are developed. The evolution of the *Improved Method* is the *MARS* (Masonry Aggregate Risk Scenarios) *procedure* for estimating economic losses in relation to the damage scenarios defined for the old city centre analysed.

The results and conclusions of what above presented are summarized in the present chapter: the answers to the objectives of the thesis, specified in Chapter 1, are described in detail. Finally, the chapter highlights the limitations of the tools and procedures developed and provide suggestions for their improvement.

8.2. Summary of the results achieved

8.2.1. Objective 1: Literature review and database development for the analyses of historic centres hit by seismic events.

The literature review focused on the analyses of the methodologies available for the seismic vulnerability assessment of historical centres. In this sense, three main approaches are identified:

- *Empirical/Expert Base Methods*: less accurate, but with a small amount of data, are able to provide a first indication of the seismic behaviour of the analysed structure;
- *Analytical/Mechanical Methods*: more reliable, but the level of details required (structural, materials, soil, etc.) does not allow their large-scale application;
- *Hybrid Methods*: try to overcome the limits of the previous methods. VULNUS and FaMIVE procedures represents and evolution of the observational approaches, but they are affected by the same uncertainties of Empirical/Expert Base methods, such as the quality of the available information.

The Empirical/Expert Base methods are identified as the most suitable approach for the assessment of seismic vulnerability of historical centres on a large scale: the GDNT (1994), the Formisano et al. (2009), the Vicente (2010) and the Ferreira et al. (2012) form are taken as reference to verify the reliability of this approach in predicting seismic damage scenarios. The principal issues of Empirical/Expert Base methods are their high dependence on the building characteristics and on the seismic scenario(s) originally used to calibrate and validate them (D'Ayala and Novelli 2010). Another limitation is related to the definition of a single value for the vulnerability, which can potentially increase the difference between the real and expected damage.

The knowledge phase of Campi Alto di Norcia, hit by the 2016 earthquake, is taken as reference to study the urban organization of a typical medieval historical centre, as well as the dynamic behaviour of its masonry aggregates. The knowledge activities proved to be very useful for the analysis of the vulnerabilities concerning the masonry aggregates, which are the result of an evolution process occurred during the centuries (Cardani 2003), (Arzilli and De Venz 2018), (Baruffaldi and Marchi 2019).

The urban organization of historical centres is determined by the morphology of the territory, the main economic activity and its strategic position, while the architectural shape of the buildings is influenced by the material available in the region and by the construction traditions. All these aspects, not strictly related to the engineering field, must be taken into account in order to guarantee a general overview of the construction typology.

As a general remark, the fragilities detected in buildings are caused by the inconsistencies between the reconstructed and preserved parts, as well as the inadequate the connections between the structural elements. Past retrofit interventions frequently increased vulnerabilities instead of reducing them (Borri et al. 2018).

8.2.2. Objective 2: Evaluation of the accuracy of different Iv methods in predicting the damage scenarios.

In general, the results of the analyses confirm the limitations learned during the literature review of these methods: i.e. their high dependence on the building characteristics and on the seismic scenario(s) originally used to calibrate and validate them. The latter influence the individual weights of considered vulnerability parameters, which, in turn, may lead to poorly accurate results. Another limitation is related to the definition of a single value for the vulnerability assessment, which can potentially increase the difference between the real and expected/ analytical damage. A range of vulnerability values, rather than a discrete value, may be a reasonable way to account for uncertainties in the assessment. The uncertainties are related to buildings' typologies, structural and architectural features and, besides, to the specific expertise and perception of the technicians performing the evaluation.

The comparison of the theoretical error with the real damage scenario revealed also another important aspect: the distribution of estimated damage is concentrated around the middle classes of the EMS-98 scale, not allowing to consider low (D0) or high (D5) damage classes, even for very low or high vulnerabilities. This leads to an increase of the relative error between the theoretical and real damage, resulting from a sort of incorrect estimation of the formulation, suggested by Bernardini et al. (2007), and not by the vulnerability estimation itself.

The GIS environment has proved to be a powerful system for data storage and management, providing also cross correlation analysis between building vulnerability and damage detected. The improvement and upgrade of the information collected can be helpful for the development of strengthening strategies and cost-benefit analyses of the case study analysed, but also for emergency planning and civil protection of similar historical centres.

Finally, the GDNT form, developed for isolated buildings, show the worst damage scenario, while the Formisano and Ferreira methods are less conservative, assigning in average lower vulnerability values to the buildings in aggregate of the case study. The Vicente form proves to be the most reliable vulnerability index method for this construction typology and, therefore, it is taken as basis for the development of the improved method.

8.2.3. Objective 3: Development of a seismic vulnerability assessment method for masonry Aggregate.

Based on the Vicente (2010) form an Improved Vulnerability Index method is developed for the construction typology of the historical centres located in Central Italy. The results show that the Improved Method can better describe the damage scenario of the case study of Campi Alto, thanks to new parameters, evaluated to take into account the out of plane behaviour of masonry buildings in aggregate. Moreover, a calibration procedure based on Bayesian Inference is adopted to reduce the relative error between the theoretical damage and the real one. The weights' calibration is performed through the damage scenario of Campi Alto hit by a seismic event with a specific intensity ($I_{EMS98} \approx 8$): the Improved Method is tested for the vulnerability assessment of historical centres only for a seismic intensity similar to that used. However, the results of the answers obtained from several experts in that field show that the level of the importance assigned mathematically to the weights is in most cases correct, ensuring greater robustness to the method. To improve the calibration of the weights

for different seismic intensities, it is possible to use the same calibration process with a bigger sample space, based on post-earthquake damage data of the same construction typology, hit by different earthquakes (Ferreira et al. 2020b).

Finally, the damage scenarios evaluated with the information quality (IQ) Index provide better results for the forecast of damage scenarios. In this sense, a range of vulnerability values, rather than a discrete value, may be a reasonable way to account for uncertainties in the assessment. The main limits of the vulnerability index methodologies can be overcome, providing more possible results, simplifying the management of seismic risk on territorial scale.

Figure 8.1 shows the process for the application of the Improved Method for the seismic vulnerability assessment of masonry aggregates on urban scale.

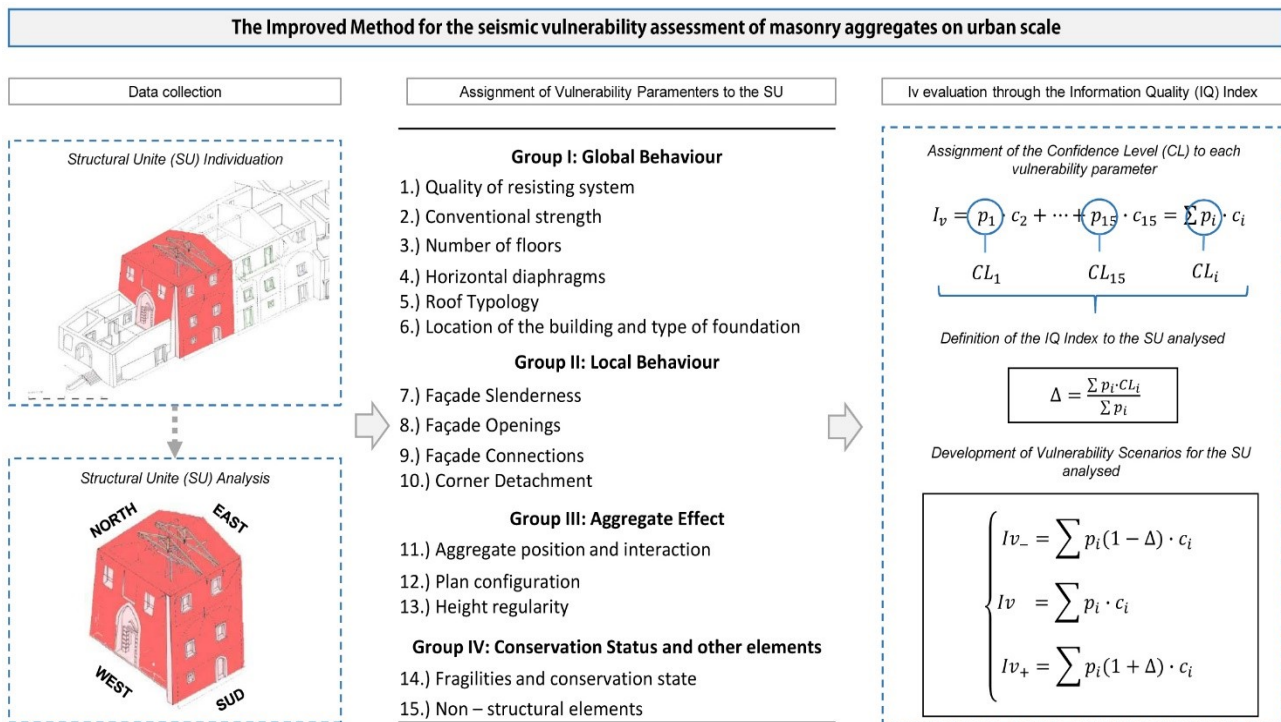


Figure 8.1: Graphical scheme that shows the application of the Improved method with the IQ index.

8.2.4. Objective 4: Definition of a codified procedure for the evaluation of economic impact of seismic events on historical centres.

The MARS procedure proposes a quick technique for the preliminary screening of seismic risk of masonry buildings in aggregates on urban scale. The results of the vulnerability assessments carried out with an Empirical/Expert base approach, such as the Improved Method, are compared with the seismic hazard of the territory in order to define the possible damage scenarios. In this sense, the MARS procedure is an evolution of the Improved Method, as it completes the vulnerability assessment of a historical centre through the expected losses.

The losses analyses carried out for the case study of Campi Alto di Norcia hit by the earthquake in Central Italy in October 2016, suggests that economic losses need less information, in order to be assessed and predicted.

For these reasons, the MARS procedure, which is a quick and simplified methodology, focuses on forecasting economic rather than human losses. A seismic risk class probability matrix is defined for the evaluation of the probable economic losses in a historic centre, based on the vulnerability of the masonry aggregates and on the seismic hazard of the territory. The matrix is calibrated through the damage scenario of Campi Alto. It is noted, that the physical damage of the case study is caused by a seismic event with a specific intensity ($I_{\text{EMS98}} \approx 8$): this means that the seismic risk probability matrix is verified only with a seismic intensity similar to that used. The performance of forecast and prediction models of the MARS procedure can be improved by increasing the sample space of the data used for the statistical analysis, in particular to take into account additional seismic intensities (Kossobokov et al. 2015). In this way, it would be possible to take advantage of the potential proposed by the MARS procedure, for the classification of the territorial seismic risk of other historical centres with the same construction typology. However, the comparison of the expected seismic risk classes, evaluated through the matrix, with the real damage observed, suggests that its application provides conservative results and therefore defines the worst case scenario of losses.

Starting from the probability of losses, suggested by the seismic risk class of the MARS procedure, it is possible to estimate the economic value of the reconstruction costs after the seismic event. The prediction of the repair costs of seismic damage is developed on the basis of damage scenarios, which occurred in a historic centre due to a seismic event, multiplying the percentage of loss with the replacement cost (Ferreira et al. 2017a). The report developed by ReLUI (2010) summarizes the reconstruction costs of the municipalities affected by the earthquake in Abruzzo on 6 April 2009 and statistically defines the average cost of repair and reconstruction of the building typology analysed. The correlation of the results of the MARS procedure with the information coming from this document can be taken as a reference, for the evaluation of the repair cost after an earthquake for the historic centres in the Apennines of Central Italy.

Finally, the theoretical estimation of damage scenarios, seismic risk classes and reconstruction costs of masonry buildings in old urban centres can be a valuable tool for the seismic risk mitigation. Indeed, by correlating the retrofit cost with the economic losses, it is possible to quantify the effect of building strengthening from an economical viewpoint and to highlight the importance of preserving traditional constructions systems compared to the cost of managing the seismic emergency and the building reconstruction following a seismic event (Maio et al. 2019).

Figure 8.2 summarizes graphically the procedure for the application of the MARS procedure, as result of all the analyses carried out and experience gained during the research activity: in the first phase, the data of the historic centre must be acquired. The latter not only takes into account the structural features of the buildings, but must provide a global overview of the urban environment analysed. All the documentation obtained, such as the maps of the urban land registry, the drawings of past retrofits and the information collected during the first inspections are used for the application of the Improved method with the IQ index. The post – processing data allows to assess the seismic vulnerability of the building stock (second phase).

The seismic hazard of the area, where the historic centre is located, and the vulnerability index of the buildings define the seismic risk class of each SU analysed and, therefore, their probability of economic loss (third phase). The calibrated seismic risk classes of the construction typology are highlighted in red.

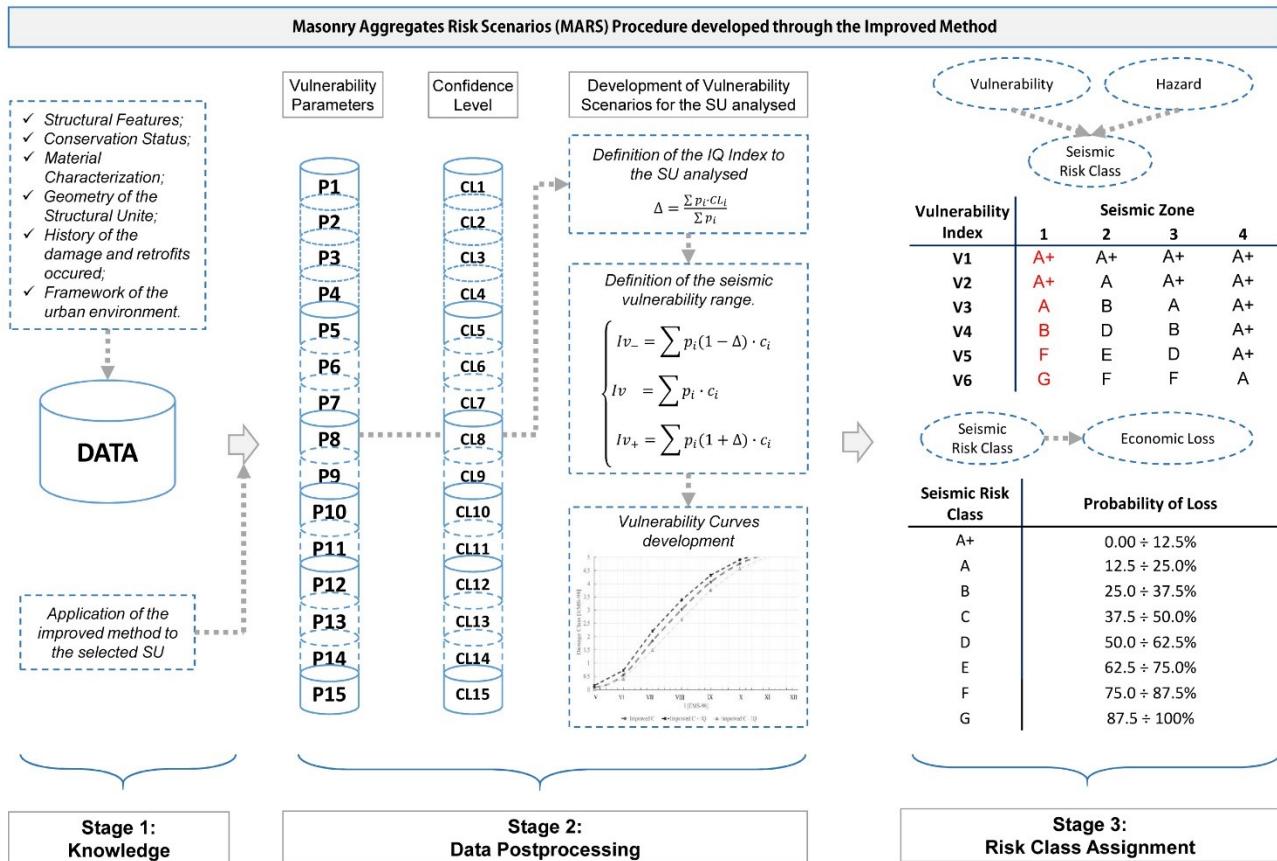


Figure 8.2: Graphical scheme that shows the application of the MARS procedure

8.3. Future research

As specified in different chapters and paragraphs of the thesis, the main issue of the tools developed for the seismic risk classification of historic centres on urban scale, is the lack of data for the calibration of the procedure for different seismic intensities. The weights' calibration is performed for a specific seismic intensity ($I_{EMS98} \approx 8.0$); this means that the proposed method, which is robust and reliable for the wide structural typologies of the Central Italy under relatively high seismic event, partially loses its efficiency with the distance from the reference condition. Still, mathematical models can be a valid tool, when calibrated basing on real data, to achieve correct predictions. In this way, vulnerability curves for this construction typology can be developed for different seismic intensities. It is notes, that this process is inevitably affected by uncertainties related to the theoretical estimation of the damage scenarios with a mathematical expression, not compared with the real damage. In this sense, the results show that the Bernardini et al. (2007) formulation must be improved, with the aim of extending the achieved results through the adoption of the calibrated model. During the analyses of the expected damage scenarios new expression for the theoretical damage were developed through a *curve fitting process* of the data available: new curves with a hyperbolic tangent or exponential shape proved to be suitable for reducing the relative error between the real and theoretical damage. The results obtained were not included in the thesis, since the development of these curves is based on good data but not enough to consider different seismic intensities. However, the analyses carried out represent a good starting point for further developments and improvements.

It is worth listing a few possibilities of future improvement of the Improved Method and MARS procedure, namely by increasing the sample space of the data used for the statistical analysis, particularly in order to take into account further seismic intensities, to consider other construction typologies and damage formulations for the definition of damage scenarios.

The work performed by Ferreira et al. (2020) on existing reinforced concrete buildings in urban centres can be taken as reference for this purpose. The information of the structural features of several masonry buildings in historic centres, as well as their damage after past seismic events, can be organized in relation to the construction typology and the seismic intensity. For each construction typology the seismic vulnerability can be assessed through the Improved Method, in order to compare the damage observed with the vulnerability index. Finally, the weights' calibration can be carried out for different seismic intensities within the context of Bayesian Inference. The regression models intend to predict the seismic capacity of masonry buildings in terms of vulnerability index, using as inputs weights and parameters of structural features. In this sense, considering the large amount of data available, the Bayesian Linear Regression can be implemented in an artificial neural network (ANN) model (Ferreira et al. 2020a). For this application, the ANN must be trained to accurately understand the relationship between the damage level and the specific vulnerability index. According to the results achieved a new analytical expression can be developed for the forecast of damage scenarios. In this sense, the issues related to Bernardini et al. (2007) formulation can be also overcome.

In addition to this, further development of the research activity consists in the definition of a Guideline for the knowledge phase of historic centres with the aim of assessing their seismic vulnerability on urban scale. The experience gained allows to identify the strategy for the survey of masonry aggregates: i.e. the identification of the aggregate from the environmental and urban context point of view, the evaluation of hypothesis on the formation and evolution of the aggregate, the geometrical and structural survey of the buildings, the identification of the material of the construction typology, definition of the crack pattern and the condition of the SUs. Furthermore, the damage detected in the case study of Campi Alto di Norcia allows to identify the causes of the structural vulnerabilities and the methodologies to solve them. Another development of the Guidelines can be focused on identifying new retrofit strategies for risk mitigation of historical centres. The new consolidation interventions must remove local and global vulnerabilities, as well as solve the vulnerabilities of past retrofit techniques, such as concrete rings not connected to masonry, that compromise the seismic performance of the buildings. Possible retrofits regard the execution of vertical ties along the buildings to avoid the vertical seismic effect (Baruffaldi and Marchi 2019), wall-to-wall connection improvement through tie-rods to ensure the “box behaviour” of the building (Ferreira et al. 2017b), the horizontal elements improvement through floor stiffening with diagonal bracing and new timber planks (Senaldi et al. 2014), wall-to-floor connection improvement, wall-to-roof connection improvement through tie-rods (Correia et al. 2015), filling the voids of multiple leaf masonry walls through grout injection techniques (Baltazar and Henriques 2015), metal ties or diatons for the transverse connection of double – leaf walls (Borri et al. 2017) and connection reinforcements of transvers piers through bindings in fibre reinforced polymers or composite reinforced mortar (Di Tommaso et al. 2017), (D’Antino et al. 2019). The price analysis of these consolidation techniques allows to define an average value of the costs per square meter of the retrofit interventions. Finally, the comparison between the expected repair costs of the historic centre due to a seismic event, evaluated with the MARS procedure, and the estimated costs for the retrofit scenario provide important information for public authorities for the management and mitigation of the seismic risk on urban scale (Maio et al. 2019).

To sum up, the present work enriches not only the knowledge of seismic analyses and protection of historic centres, but also provides several future lines of research. In a society, where human and economic resources are few and scattered, the development of quick but accurate methods for the analyses of seismic vulnerability of existing buildings is the most practical way to mitigate the seismic risk. The importance of these studies not only provides a safer built environment but guarantees the protection of old historic centres, which represents the identity of local communities and the national cultural heritage, whose preservation over time for future generations is a duty for the current ones.

8.4. References

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ANNEX A. Survey Sheet

The present Annex shows the survey sheet adopted for the collection of the data of the masonry buildings and aggregates during the site inspections carried out in Campi Alto di Norcia. These survey sheets summarize and organize in a standard way the information: in the first part, the position of the building in the historical centre is evaluated through aerial photos and plans and an identification number is assigned. In the second part, the principal structural features are defined: masonry characteristics, roof and floor typologies, presence of retrofits, structural details (i.e. vaults, stairs, etc.). A series of drawings of the plans and façades of the building are shown with their geometric values. In the last part, an estimation of the possible weights of the floors is performed, based on the information collected in the previous sections.

The information achieved during the in situ surveys allowed to apply the selected methods with a certain level of confidence and made the obtained results robust. In the following pages the survey sheet of the SU 197 is shown as an example.

FORM FOR GEOMETRIC AND SEISMIC DAMAGE SURVEY OF THE BUILDINGS

ID Form: 35	Historical centre:	Campi Alto di Norcia
Date of the survey: 14.02.2019	Building No. (Land Registry):	SU_197
Technician: Eng. Federico Romis	Address: Via Dell'Aquila	
Property: XXX		Tel.: XXX
Reference for the inspection: XXX		Tel.: XXX
Designer of the building and / or of the retrofits: XXX		Tel.: XXX

East View



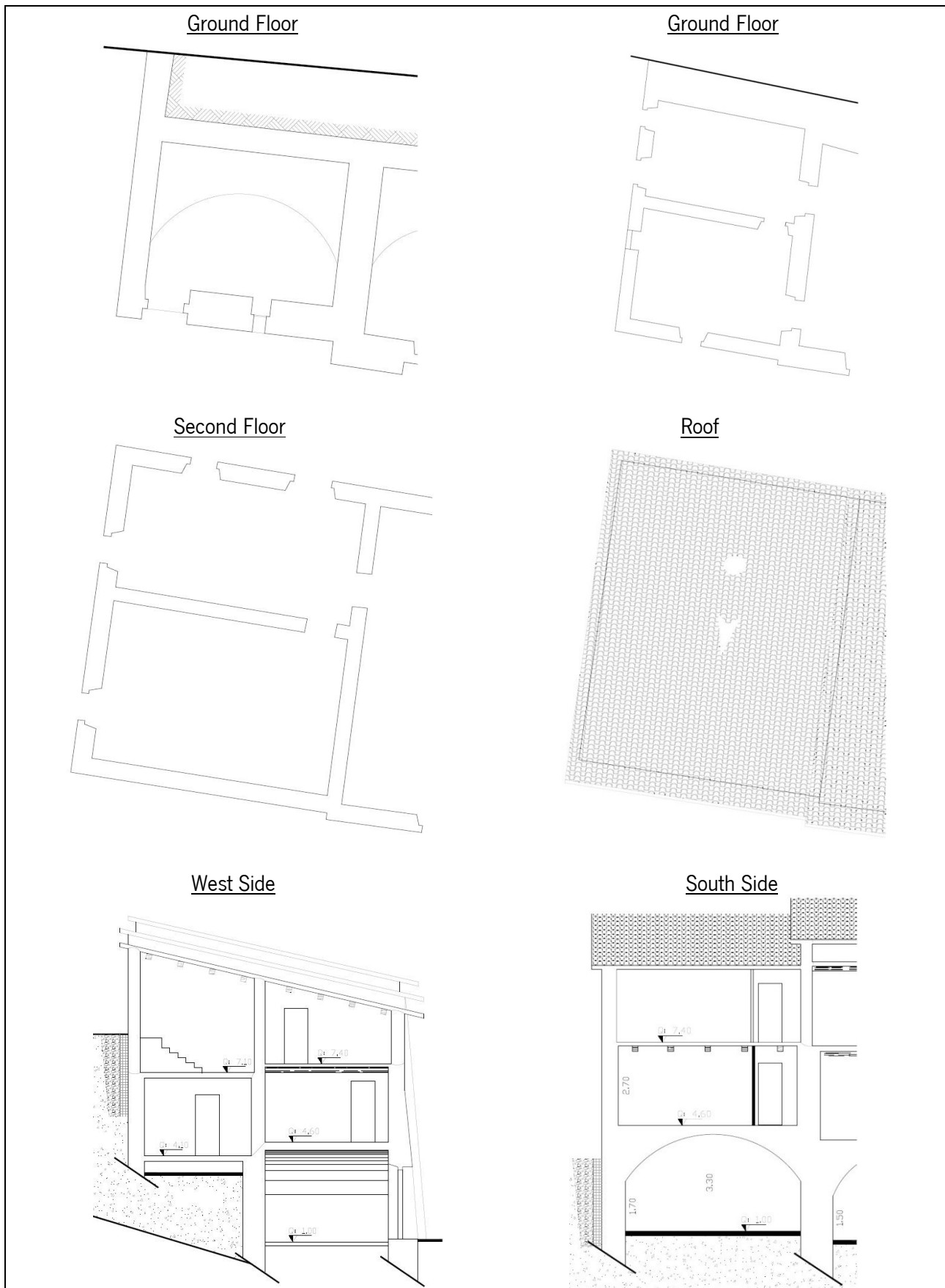
South View

**LOCATION OF THE BUILDING**

Section 1: Survey of the building

Position of the SU: ☐ isolated ☒ connected with other SUs on N° of sides: 1 Connection typology: ☐ internal ☒ extreme ☐ corner	Construction Typology: ☐ urban architecture ☒ rural architecture ☐ religious architecture ☐ school architecture ☐ industrial architecture ☐ hospital architecture ☐ military architecture ☐ Other:	Property: ☐ Public ☒ Private Intended Use: ☒ residential ☐ productive ☐ commercial ☐ religious ☐ agrarian ☐ public services ☐ storage ☐ Other:	Construction Age Modifications: ☒ Yes (Year – Technical Code): 2003 – Post seismic 1997 ☐ No Conservation Status: ☒ Good ☐ Poor ☐ Bad ☐ Ruin ☐ Collapse
Location characteristics: ☐ on a plane ☒ on a slope ☐ on natural relief ☐ on artificial relief ☐ depression	☐ Tower ☐ Palace ☐ Castle ☐ Church ☐ Other:		
Geometry: ☒ square ☐ rectangular ☐ polygonal ☐ other	Number of floors: ☒ above ground: 2 ☐ basement: ____ <u>Dimensions in square meters (mq):</u> Internal: 4.6x6.00 <u>Dimensions in meters (m):</u> Floor height: 3.30 (PT); 2.70 (P1) ; 3.20 (P2) Roof height: 8.20	Stairs: ☐ external ☐ internal ☐ attached ☒ no	Material: ☐ wood ☐ steel ☒ stone ☒ brick ☒ vault ☐ concrete
Roof: ☐ Wood Thrust ☒ Wood no Thrust ☐ Wood Truss ☐ Concrete or steel Retrofits: ☒ Concrete ring ☐ Bracing ☒ Other: Concrete slab	Vertical structure: ☐ disordered masonry stone ☒ rough masonry stone ☐ regular masonry stone ☐ irregular masonry (tuff, etc) ☐ regular masonry (tuff, etc) ☐ masonry bricks and lime mortar ☐ masonry bricks and cement mortar Retrofits: ☐ reinforced plaster ☒ injections ☒ improved mortar joint ☐ new wall face ☐ buttress ☒ wall connection improvement ☐ Other:	Horizontal structure: <u>Vault:</u> ☒ barrel ☐ cruise ☐ pavilion <u>Floors:</u> ☒ wood ☐ beams and vaults ☐ beams and bricks ☒ concrete ☐ steel Retrofits: ☐ metal ties ☐ concrete ring ☒ concrete reinforcement ☐ Other: Retrofits: ☒ metal ties ☒ concrete rings ☐ concrete slab ☐ bracing ☐ stiffening ☐ Other:	

Section 2: Drawings



Section 3: Static Loads**First Floor****Typology: stone barrel vault****Geometry**

Width [m]:	4.60
Length [m]:	6.00
Height [m]:	3.30
Vault height [m]:	1.40
Vault thickness [m]:	0.15
Floor thickness [m]:	0.15

G1 (permanent structural loads):

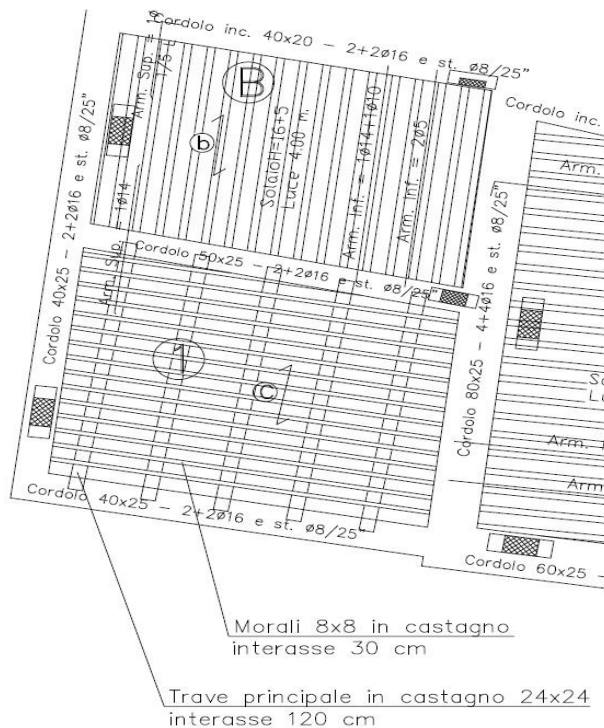
Stone Vault [KN/mc]	21.00
Concrete Layer [KN/mq]	1.00

G2 (permanent no structural loads):

Sand/cement layer (10cm) [KN/mq]	1.60
Wood slab (0.04m) [KN/mq]	0.64
Terracotta floor [KN/mq]	0.30

Ok (Overloads)

Cat. A (residential) [KN/mq]	2.00
------------------------------	------

Second Floor**Typology: concrete floor****Geometry**

Width [m]:	4.00
Length [m]:	6.20
Slab thickness [cm]:	16+5
Floor thickness [cm]:	10.00

G1 (permanent structural loads):

Concrete slab [KN/mq]	2.00
-----------------------	------

G2 (permanent no structural loads):

Sand/cement layer (10cm) [KN/mq]	1.60
Terracotta floor [KN/mq]	0.30

Ok (Overloads)

Cat. A (residential) [KN/mq]	2.00
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Typology: wood floor**Geometry:**

Cordolo inc. 40x20 - 2+2ø16 e st. ø8/25" Solai di Copertura in legno massiccio Trave principale in castagno 24x24 interasse 120 cm Morali 8x8 in castagno interasse 30 cm	<table> <tr> <td>Width [m]:</td><td>4.60</td></tr> <tr> <td>Length [m]:</td><td>6.2</td></tr> <tr> <td>Slab thickness [cm]:</td><td>24+8</td></tr> <tr> <td>Floor thickness [cm]:</td><td>10.00</td></tr> <tr> <td>G1 (permanent structural loads):</td><td></td></tr> <tr> <td>Wood beam 24x24 [(i=120cm) KN/mc]</td><td>8.00</td></tr> <tr> <td>Wood beam 8x8 (i=30cm) [KN/mc]</td><td>8.00</td></tr> <tr> <td>G2 (permanent no structural loads):</td><td></td></tr> <tr> <td>Wooden plank (4cm) [KN/mq]</td><td>0.64</td></tr> <tr> <td>Sand/cement layer (10cm) [KN/mq]</td><td>1.60</td></tr> <tr> <td>Terracotta floor [KN/mq]</td><td>0.30</td></tr> <tr> <td>Ok (Overloads)</td><td></td></tr> <tr> <td>Cat. A (residential) [KN/mq]</td><td>2.00</td></tr> </table>	Width [m]:	4.60	Length [m]:	6.2	Slab thickness [cm]:	24+8	Floor thickness [cm]:	10.00	G1 (permanent structural loads):		Wood beam 24x24 [(i=120cm) KN/mc]	8.00	Wood beam 8x8 (i=30cm) [KN/mc]	8.00	G2 (permanent no structural loads):		Wooden plank (4cm) [KN/mq]	0.64	Sand/cement layer (10cm) [KN/mq]	1.60	Terracotta floor [KN/mq]	0.30	Ok (Overloads)		Cat. A (residential) [KN/mq]	2.00								
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	<u>Qk (Overloads)</u>	
	Cat. H (roof) [kN/mq]	0.50

ANNEX B. Retrofits Typologies

The present Annex shows the retrofits techniques detected in the structural aggregates (SA) of Campi Alto di Norcia, where the interventions detected on site during different inspections are highlighted with a red cross, while the interventions reported by the retrofitting projects presented to the Umbria Region, after 1979 and 1997 seismic events, are identified with a black cross.

These are divided into two main categories in relation to their execution, i.e. before or after the Valnerina earthquake of the 1979 (*old/modern retrofit*). The most common operations, in the first case, were the addition of masonry buttress to the external wall of the SU, the introduction of metal ties to connect the opposite wall facades of the buildings, etc. Modern retrofit – performed even after the 1997 Umbria-Marche seismic event, consisted in the replacement of timber roofs and timber floors with concrete slabs, in the application of injections and steel stitching to fill the voids in the masonry walls and to connect different masonry faces. Concrete plates to connect the walls with the new floors and roofs are also common, as well reinforced cement-based plaster to reduce the irregularity of the masonry composition.

The position and the analysis of the adopted techniques allowed to know the structural characteristics of the SUs and its dynamic behaviour, therefore helped the application of the Iv methods in the case study. The comparison between the retrofits intervention and the damage detected shows its performance during a seismic event.

	Classic Retrofit interventions					“Modern” Retrofit interventions (1982 – 2003)													
Structural Aggregate	Buttress	Metal ties	Wood slab remake	Masonry reinsection	Filling cracks	Mortar Joint	Concrete acritrave	Wall rebuild	Top Concrete ring	Storey Concrete ring	Half Concrete ring	Foundation concrete ring	Masonry Injection	Reinforced plaster	Concrete cover	Storey stiffening	Reinforced perforations	Reinforced vault	Building remake
SA 1	X					X		X	X				X		X			X	
SA 2																			
SA 3	X					X	X	X	X			X	X		X		X	X	X
SA 4	X					X		X	X			X	X		X	X		X	
SA 5	X		X						X						X				
SA 6																			
SA 7	X	X				X			X						X	X			
SA 8	X					X			X						X				
SA 9				X		X			X	X			X		X	X			
SA10	X														X				
SA11								X	X						X	X		X	
SA12	X							X	X	X					X			X	
SA13	X			X		X	X	X	X	X			X	X	X	X		X	
SA14	X					X		X	X	X			X	X	X				
SA15	X					X			X			X	X	X	X	X		X	
SA16	X					X									X				
SA17	X									X		X			X				
SA18	X							X	X				X		X	X		X	
SA19	X					X		X	X				X		X	X		X	
SA20						X	X	X	X				X		X	X		X	
SA21	X	X	X		X	X	X	X	X		X	X	X		X	X		X	
SA22	X		X		X			X	X	X			X		X	X		X	
SA23	X	X		X		X	X	X	X		X		X	X	X	X	X	X	
SA24	X	X	X			X			X									X	
SA25	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	
SA26	X	X				X			X						X			X	
SA27	X	X	X			X	X	X	X				X		X	X		X	X
SA28	X	X				X	X	X	X				X	X	X	X		X	
SA29	X	X				X			X						X				
SA30						X	X	X					X		X			X	
SA32	X					X		X					X		X			X	
SA33						X	X	X					X		X			X	
SA NEW																			X

ANNEX C. Guidelines for the application of the Improved Method

The present Annex presents the guidelines prepared for an easier application of the Improved Method and the fulfilment of the survey sheet. The latter is composed by five main groups, in which the 15 vulnerability parameters are distributed, describing the structural to take into account for the analyses of dynamic behaviour of the structural unit in the masonry aggregate. The guidelines for the application of the first eleven parameters comes from the Vicente method, while the last four parameters were developed within the research activity.

The suggestions proposed by Vicente are used as reference and enriches with the experience gained during the research activity performed in the case study. According to the procedure proposed the final Vulnerability index is reduced or increased through the IQ Index, which uncertainty scale is reported for the application of the improved method.

VULNERABILITY INDEX OF THE STRUCTURAL UNIT			
ID Form:		Historical centre:	
Date of the survey:		Building No. (Land Registry):	
Technician:		Address:	
Property:			Tel.:
Reference for the inspection:			Tel.:
Designer of the building and / or of the retrofits:			Tel.:

VULNERABILITY PARAMETERS		VULNERABILITY CLASS (C)				WEIGHT (P)	C, P	UNCERTAINTY LEVEL (UL)	COMMENTS
Group I: Global Behaviour									
P1	Quality of resisting system	A	B	C	D	2.00			
		0	5	20	50				
P2	Conventional strength	A	B	C	D	2.00			
		0	5	20	50				
P3	Number of floors	A	B	C	D	0.50			
		0	5	20	50				
P4	Roof typology	A	B	C	D	1.00			
		0	5	20	50				
P5	Floor typology	A	B	C	D	3.00			
		0	5	20	50				
P6	Location of building and type of foundation	A	B	C	D	0.75			
		0	5	20	50				
Group II: Local Behaviour									
P7	Façade Slenderness	A	B	C	D	3.00			
		0	5	20	50				
P8	Façade Openings	A	B	C	D	0.75			
		0	5	20	50				
P9	Façade Connections	A	B	C	D	0.50			
		-20	-10	-5	0				
P10	Corner Detachment	A	B	C	D	2.00			
		0	5	20	50				
Group III: Aggregate Effect									
P11	Aggregate position and interaction	A	B	C	D	1.00			
		0	5	20	50				
P12	Plan configuration	A	B	C	D	0.75			
		0	5	20	50				
P13	Height regularity	A	B	C	D	0.75			
		0	5	20	50				
Group IV: Conservation Status and other elements									
P14	Fragilities and conservation status	A	B	C	D	1.00			
		0	5	20	50				
P15	Non-structural elements	A	B	C	D	1.00			
		0	5	20	50				

VULNERABILITY INDEX APPLICATION		
Vulnerability Index (I_v normalized, $0 < I_v < 100$)	$0 \leq I_v = \sum_{i=1}^{15} p_i \cdot c_i \leq 975$	
Vulnerability Index range (I_v normalized, $0 < I_v < 100$)	$I_v = \begin{cases} I_v^- = \sum p_i(1 - \Delta) \cdot c_i & (\text{lower bound}) \\ \overline{I_v} = \sum p_i \cdot c_i & (\text{median value}) \\ I_v^+ = \sum p_i(1 + \Delta) \cdot c_i & (\text{upper bound}) \end{cases}, \text{ with } \Delta = \frac{\sum p_i \cdot UL}{\sum p_i}$	
UNCERTAINTY SCALE		
Confidence Level	Description	Uncertainty Level (UL)
HIGH CONFIDENCE (H)	HIGH INFORMATION QUALITY: Certain Reliability of the data because of direct measured of the entire building.	0.0 %
HIGH - MEDIUM CONFIDENCE (HM)	HIGH MEDIUM INFORMATION QUALITY: Good Reliability of the data because of direct/indirect measured of the entire building.	10 %
MEDIUM CONFIDENCE (M)	MEDIUM INFORMATION QUALITY: Medium Reliability of the data because only indirect measured in-situ of the building.	20 %
MEDIUM - LOW CONFIDENCE (ML)	MEDIUM - LOW INFORMATION QUALITY: Medium-Low Reliability of the data because indirect measured non-in-situ of the building.	30 %
LOW CONFIDENCE (L)	LOW INFORMATION QUALITY: Low Reliability of the data because of measures evaluated on the basis of hypotheses verified randomly in the building or in similar building typologies in the same urban settings	40 %
LOW - ABSENT CONFIDENCE (LA)	LOW - ABSENT INFORMATION QUALITY: Very Low Reliability of the data because of measures evaluated on the basis of hypotheses non verified in the building and in the urban setting.	50 %
ABSENT CONFIDENCE (A)	ABSENT INFORMATION: NO reliability	60 %

PARAMETER 1: Quality of resisting systems

This parameter evaluates the quality of the resisting system, towards the homogeneity, dimension and nature of the material, the composition and arrangement of masonry and the type of cross-section within the wall (i.e. double leaf wall, etc.). The first aspect to identify is the origin of material: the identifications of the quarries nearby, the preparation process of mortar and the wood typology used, are the first information to be acquired for the definition of the mechanical characteristic of the material. After that, the construction typology is analysed, looking to the regularity and disposition of stones or bricks, as well as their homogeneity, and the thickness of

the mortar joints. The presence of elements (diatones) across the two faces of the wall and the presence of stones with greater dimensions at the corners or openings of the building is also evaluated. It is notes, that the external face of a thick wall or double leaf wall can be built with a more careful selection and a better device of the bricks or stones, than the internal face. It is, therefore, strongly recommended to view both sides of the walls and possibly remove the peace of the plaster on the inside to have a clear overview.

The classification of the GDNT-SSN (1994) is introduced to define the vulnerability classes of the quality of resisting system. The latter takes into account, in an indirect and qualitative way, the homogeneity and mechanical properties of masonry walls.

Table C. 1: Definition of the vulnerability classes for parameter P1

Description	Class	Score
Good quality brick (solid or perforated < 45% voids), well-cut stone or tuff masonry wall, with connections between the two faces and with a homogeneous distribution throughout the structural system.	A	0
Brick (perforated area <45%), well-cut stone masonry with different units throughout the length of the walls, which reduces its homogeneity. Irregular stone masonry with cross connection between the two faces of the wall.	B	5
Low quality brick masonry with irregularities in the disposition. Irregular stone masonry with not squared units and heterogeneous dimensions. Absence of transversal connection in the double leaf wall, however masonry is made by good mortar and well joint.	C	20
Poor quality brick masonry with stone fragments. Stone masonry with very irregular units without careful locking (presence of voids). Irregular stone masonry without cross connection, poor mortar and poorly joint.	D	50

Figure C. 1, Figure C. 2, Figure C. 3 and Figure C. 4 shows the masonry typologies for each vulnerability class, as defined by Vicente (2010).

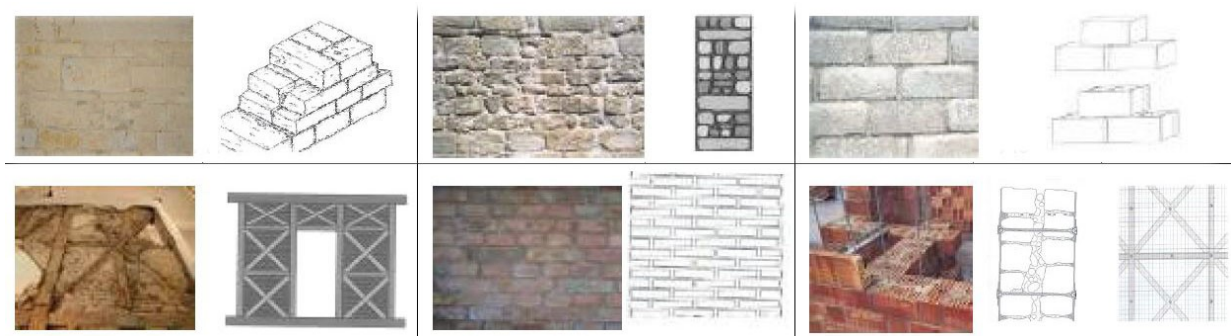


Figure C. 1: Masonry typologies identified for class A.

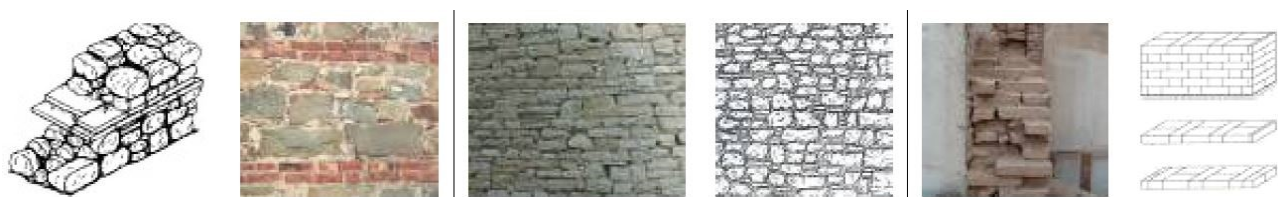


Figure C. 2: Masonry typologies identified for class B.

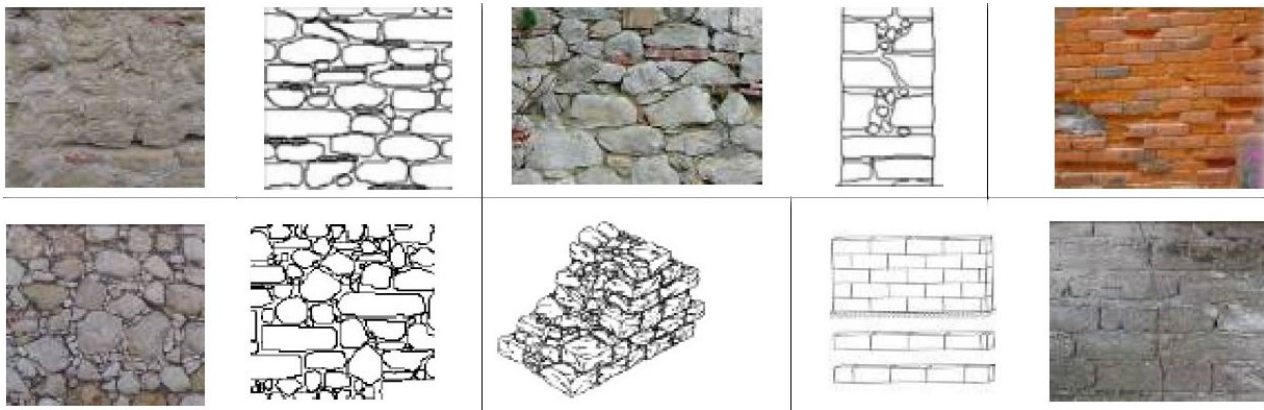


Figure C. 3: Masonry typologies identified for class C.



Figure C. 4: Masonry typologies identified for class D.

The building and retrofitting period provides important information on the techniques used for the masonry construction and reinforcement, allowing to improve or reduce the class of parameter 1. According to Vicente method (2010), Table C. 2 offers further indication for the correct assignment of the classes, based on the experience achieved during the damage detection of the last earthquake events occurred in Central Italy. In this sense, the interventions are divided in two main groups: the seismic reinforcement (i.e. fibre meshes on one or both sides of the wall or/and transverse connections through tie rods in double leaf walls) and the masonry improvement (i.e. injections to reduce the voids, restoration of mortar joints).

Table C. 2: Class assignment according to the building year and retrofits techniques.

First Vulnerability class assigned	Building Year			Retrofit techniques	
	<1900	1900 - 2000	>2000	Seismic reinforcement	Masonry consolidation
A	B	B	A	A	A
B	C	D	B	A	A
C	D	D	C	B	B
D	D	D	D	B	C

PARAMETER 2: Conventional strength

Under the hypothesis of global “box-behaviour”, this parameter provides an estimation of the resistance of a masonry building to the horizontal actions, through a simplified formulation that compares the building, in its weaker direction, to an equivalent shear wall (C). The method itself notes, that the reliability of the results decreases for buildings built before 1900, which are not retrofitted, and where the global “box-behaviour” is not guaranteed. The results achieved must, therefore, should be adopted with extreme caution. The shear resistance

is evaluated through the formulation of Turnšek and Cacovic (1971), as a function of the characteristic tangential stress of the masonry wall (τ_k) and the average vertical compressive load σ_o , which bears on it. For its evaluation it is, therefore, necessary to define for each plan:

- The number of floors starting from the ground floor (N);
- The average covered area over the section that identify the verification plan (A_i);
- The total cross sectional areas of resistance elements in the two orthogonal directions (A_x, A_y);
- The characteristic tangential shear resistance of the masonry type of the building (τ_k);
- The average height of each storey (h);
- The average weight, per unit of covered area, of a building level, as the sum of the weight of a slab and a wall within a floor (q);

The walls to take into account as resisting elements (A_x, A_y) should have continuity in height of the building. In the case of oblique walls, in relation to the two directions defined main areas, the resistant area in each direction is projected in the direction of each main axis (multiplied by $\cos(2\alpha)$, where α being the value of the angle of deviation in relation to the main axes). In case of a wall between to SUs, the resistant area considered in the analysis of the singular building is only half.

In the absence of information about the mechanical properties of the masonry of the building analysed, the determination of the shear resistance can be assessed through the quality of resisting system (P1): the bibliography database, developed for example by ReLuis, allows to associate a mechanical value to the masonry typology identified, thanks to several statistical data performed in the last decades. The average weight (q) can be evaluated, in relation to the average specific weight of the masonry (p_m), the average weight per unit area of the floor (p_s) and the average of the floor height (h), with the following expression [C.1]:

$$q = \frac{(A + B)h}{A_t} \cdot p_m + p_s \quad [C.1]$$

Finally, the average shear resistance is evaluated with formulation [C.2]:

$$C = \frac{a_o \tau_k}{qN} \sqrt{1 + \frac{qN}{1.5 a_o \tau_k (1 + \gamma)}} \quad [C.2]$$

Where a_o is the ratio between the minimum of the cross sections ($A = \min [A_x, A_y]$) and A_i , while γ is the ratio among the maximum cross sections ($B = \max [A_x, A_y]$) and A_i .

The shear resistance is analysed at the level with the less resistance to horizontal actions, which is usually the first floor. However, there are some exceptions, such as buried floors that are totally or partially confined by the ground. In this case, the floor to be considered is the first one above the ground (Figure C. 5).

In case of height variation in terms of quality of the vertical resisting system or the presence of arcades at different levels, other cross sections must be considered (Figure C. 6).

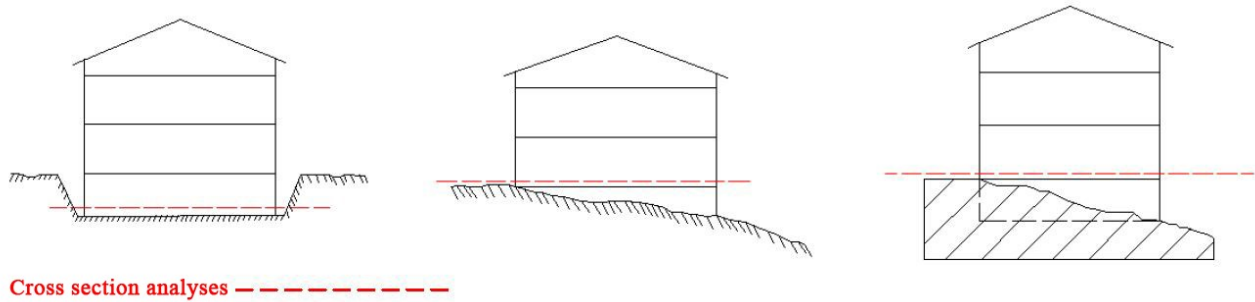


Figure C. 5: Examples of cross section analyses.

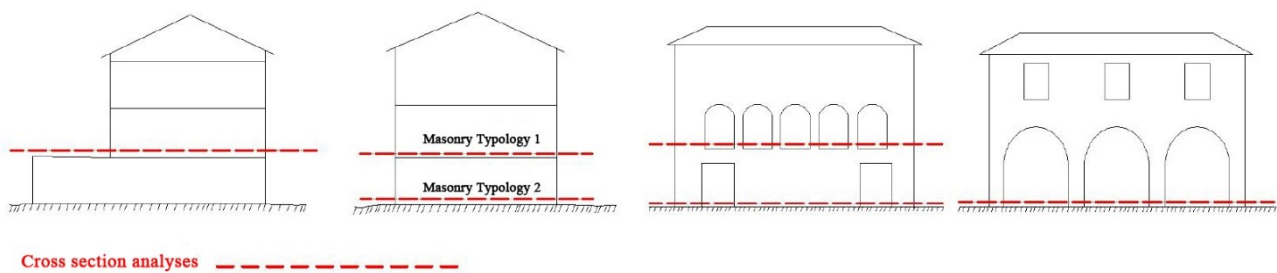


Figure C. 6: Examples of cross section analyses.

The attribution of a building to one of the four classes takes place on the basis of the ratio $\alpha = C/\bar{C}$ between the value of C , obtained as mentioned above, and the value of $\bar{C}$ assumed equal to 0.4.

Table C. 3: Definition of the vulnerability classes for parameter P2

Description	Class	Score
Building with a conventional strength $\alpha \geq 1.0$	A	0
Building with a conventional strength $0.6 \leq \alpha < 1.0$	B	5
Building with a conventional strength $0.4 \leq \alpha < 0.6$	C	20
Building with a conventional strength $0.4 < \alpha$	D	50

PARAMETER 3: Number of floors

The criteria of the parameter suggest that taller masonry buildings tend to be more vulnerable and susceptible to second-order effects, when compared to low rise structures. This parameter doesn't assess the height irregularity, nor associate the stiffness of the building with its height, but rather expose the concept of relativity.

Table C. 4: Definition of the vulnerability classes for parameter P2

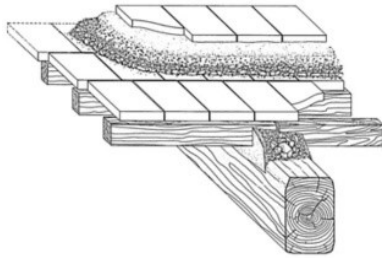
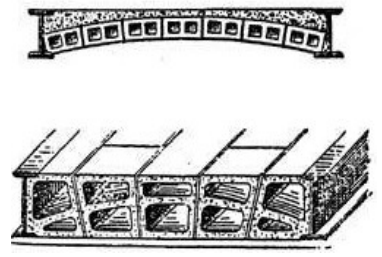
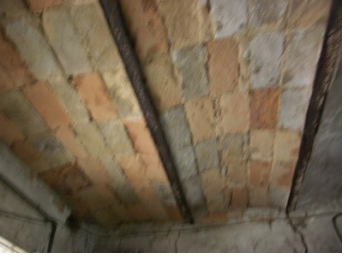
Description	Class	Score
Building with 1 floor.	A	0
Building with 2 or 3 floors.	B	5
Building with 4 or 5 floors.	C	20
Building with more than 6 floors.	D	50

PARAMETER 4: Floor typology

This parameter defines the role of the floors in relation to the global “box-behaviour” of the building, considering the connections to the vertical walls, the stiffness and the orientation of the slabs.

Regarding to the connection typology of horizontal diaphragm to masonry walls, it is notes that good solutions is: a double plank wooden floor, linked to a continuous steel or wooden support along the perimeter of the wall and joint to the latter through metal bars. On the contrary, concrete or steel slabs connected to masonry walls by concrete rings do not provide a good connection, especially if the latter isn't done for the entire thickness of the wall and joined to the masonry by metal bars. A good connection to the masonry wall, whether punctual or continuous, must be carried out in the floors along their perimeter. Regarding the slab stiffness, good solutions are: a wooden structure consisting of a double wooden plank arranged diagonally or reinforced concrete slab suitable and compatible in terms of thickness, weight and rigidity to the structural masonry. If there is more than one type of floor in the SU, it will be considered the worst situation in the assignment of the scores to the parameter. The conservation status of the floor and its connection (degradation by biological action), is taken into account by increasing the class of this parameter. To better understand and assist the assignment of the vulnerability classes of this parameter, several schemes are shown to identify the different floor typologies (**Errore. L'origine riferimento non è stata trovata.** and Table C. 12)

Table C. 5: Floor typologies according to their stiffness

Deformable floors Wooden floor with a single wooden plank with a thin concrete layer. Cross or pavilion vaults (without reinforcement) belong to this typology.	 
Semi – Rigid floor Wooden floor framework with double crossed wooden plank or made by steel/concrete beams with bick vaults and stiffened with a thin concrete layer. Barrel or thick vaults (30cm brick/stone) belong to this typology.	 

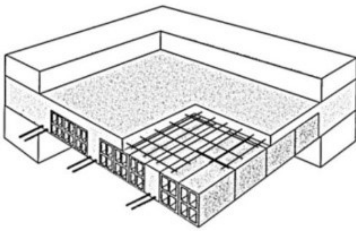
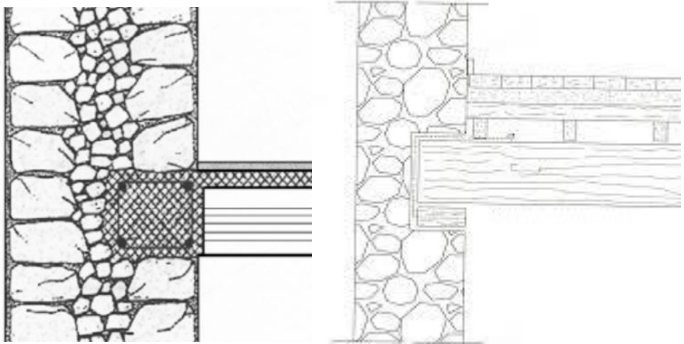
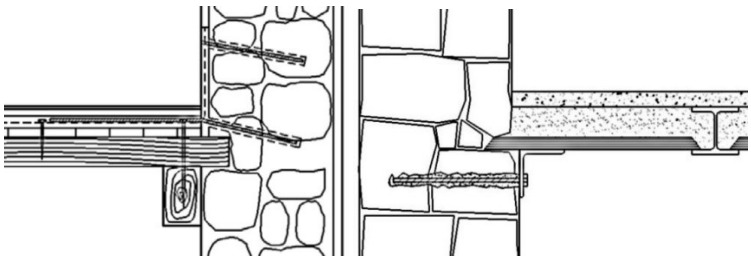
Rigid Floor Concrete or steel beams with bricks reinforced with a tick concrete layer. The thickness of these floor typologies is the biggest one.	 
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Table C. 6: Connection typologies between vertical and horizontal structures

Bad connection to the main structure Connection to masonry walls through concrete rings not build for the entire thickness of the wall and not joined to the masonry by metal bars. Wooden beam inserted in the masonry wall, without a connection along the perimeter.	
Good connection to the main structure Continuous steel or wooden support along the perimeter of the wall and/or presence of steel bars to link the horizontal element to the masonry wall.	

The typology of masonry wall and its behaviour during a seismic event is important for the classification of this parameter: if the resistant structure of the masonry walls (parameter P1) is identified with Class C, the vulnerability class for the floor typology will be D

Table C. 7: Definition of the vulnerability classes for parameter P9

Floor typology	Connection typology	Conservation Status	Class	Score
Rigid or semi rigid	Good	Good	A	0
	Bad	Good	B	5
	Good	Bad	C	20
	Bad	Bad	D	50
Deformable	Good	Good	C	20
	Bad	Good	D	50
	Good	Bad	D	50
	Bad	Bad	D	50

PARAMETER 5: Roof typology

The parameter evaluates the thrust action of the roof on the perimeter walls, the effectiveness connection among the roof structure and the walls, the weight of the roof, in terms of seismic mass, and the difference in stiffness and strength compared to the masonry under the roof. In this sense, the scores are assigned in relation to the presence or absence of concrete rings or metal ties, to the permanent load of the roof and to the pitch slope (**Errore. L'origine riferimento non è stata trovata.**).

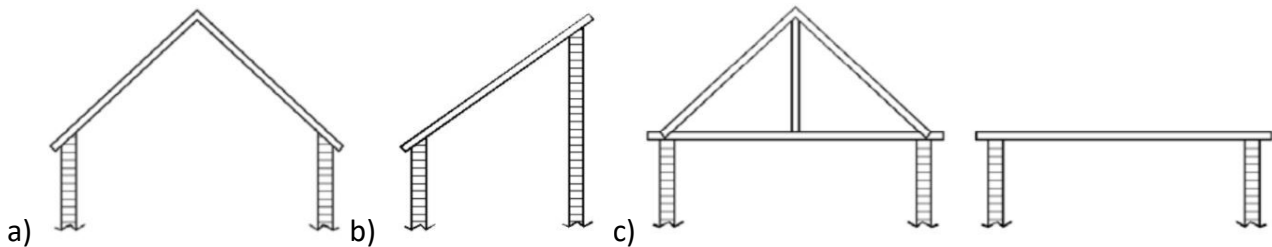


Figure C. 7: Different thrust exerting roofing structural system (a), semi-thrusting (b) and no thrusting roof types

The typology of masonry wall and its behaviour during a seismic event is important for the classification of this parameter: if the resistant structure of the masonry walls (parameter P1) is identified with Class C, the vulnerability class for the roof typology will be D.

Table C. 8: Definition of the vulnerability classes for parameter P8

Thrust Condition	Concrete Ring	Metal Ties	Conservation Status	Class	Score
No Thrust system	Yes	Yes/No	Good	A	0
	Yes/No	Yes	Good	A	0
	No	no	Good	B	5
	Yes	Yes/No	Bad	B	5
	Yes/No	Yes	Bad	B	5
	No	no	Bad	C	20
	Yes	Yes/No	Bad	C	20
	Yes/No	Yes	Worst	C	20
	No	no	Worst	C	20
	Yes	Yes/No	Worst	D	50
Less Thrust system	Yes/No	Yes	Worst	D	50
	Yes	Yes/No	Good	B	5
	Yes/No	Yes	Good	B	5
	No	no	Good	C	20
	Yes	Yes/No	Bad	C	20
	Yes/No	Yes	Bad	C	20
	No	no	Bad	C	20
	Yes	Yes/No	Bad	D	50
	Yes/No	Yes	Worst	D	50
	No	no	Worst	D	50
Thrust system	Yes	Yes/No	Worst	D	50
	Yes/No	Yes	Worst	D	50
	No	no	Good	D	50
	Yes	Yes/No	Bad	D	50

Yes/No	Yes	Bad	D	50
No	no	Bad	D	50
Yes	Yes/No	Bad	D	50
Yes/No	Yes	Worst	D	50
No	no	Worst	D	50
Yes	Yes/No	Worst	D	50
Yes/No	Yes	Worst	D	50

PARAMETER 6: Location of building and type of foundation

It assesses the location of the building, in relation to the morphology of the surrounding area, and estimates the foundations characteristics, according to the soil typology, the natural slope and the possible presence of different levels. Since it is difficult to evaluate the ground-building interaction for each condition, some procedures are introduced, to help the fulfilment of the survey sheet during the inspection:

- Assume the most correct classification possible (visual observation), adopting for all buildings in the study area with the same vulnerability class for this parameter;
- In cases of greater doubts, adopt the lowest confidence level for the IQ Index;
- Supporting the score assignment of this parameter through information coming from the building typology, geological and geotechnical characteristics of each site or zone, avoiding a case-by-case inspection.

The results provided by this parameter are less reliable for a set of buildings located in an area with different terrain morphology.

Table C. 9: Definition of the vulnerability classes for parameter P4

Ground and foundation	Slope of the ground (%)	Height difference within the foundation (m)	Class	Score
Rock (type A soil) with or without foundation	$p \leq 10$	-	A	0
	$10 \leq p \leq 30$	-	B	5
	$30 \leq p \leq 50$	-	C	20
	$p > 50$	-	D	50
Soil type B and C with foundation on rock	$p \leq 10$	$\Delta h = 0$	A	0
	$p \leq 10$	$0 < \Delta h \leq 1$	B	5
	$10 \leq p \leq 30$	$\Delta h \leq 1$	B	5
	$30 \leq p \leq 50$	$\Delta h \leq 1$	C	20
	$p > 50$	-	D	50
	-	$\Delta h > 1$	D	50
Soil type B and C without foundation	$p \leq 10$	$\Delta h = 0$	A	0
	$p \leq 10$	$0 < \Delta h \leq 1$	B	5
	$10 \leq p \leq 20$	$\Delta h \leq 1$	B	5
	$10 \leq p \leq 30$	$\Delta h \leq 1$	C	20
	$p > 50$	-	D	50
	-	$\Delta h > 1$	D	50
Soil type D and E with foundation	$p \leq 50$	$\Delta h \leq 1$	C	20
	$p > 50$	-	D	50

Soil type D and E without foundation	-	$\Delta h > 1$	D	50
	$p \leq 30$	$\Delta h \leq 1$	C	20
	$p > 30$	-	D	50
	-	$\Delta h > 1$	D	50

In the case of a building on rock (Type A), the height difference within the foundation is overlooked. The extension of the masonry wall in the ground cannot be classified as a foundation. However, it is assumed that the increase of the wall thickness at the base of the buildings can be classified as a foundation.

Information about the geophysical recognition (soil geology, stratification, SPT tests or some characterization parameters: N_{SPT} , C_u and $V_{s,30}$) allow a more detailed identification of the soil type. The name used for the soil type is proposed in EC8 § 3.1.2 Table 3.1 [CEN, 2004]. The class assignment is made in relation to the worst conditions identified.

Table C. 10: Ground class according to EC8 § 3.1.2 Table 3.1 [CEN, 2004]

Ground type and description	$V_{s,30}$	N_{SPT}	C_u
A: Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B: Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C: Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D: Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E: A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s > 800$ m/s.			
S₁: Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content	<100	\	10-20
S₂: Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S ₁			

PARAMETER 7: Façade slenderness

Since the slenderness is defined as the ratio between the height (H) of the façade and its thickness (b) at the base ($FS = H/b$), the analysis is carried out by changing the thickness of the façade benchmark. The evaluation of the masonry quality must be taken into account, in order to consider the failure of the masonry and the retraction of the overturning point (hinge) during the rotation of the façade, due to the exceeding the compressive strength of the masonry. The façade slenderness classification is, therefore, evaluated for a regular arrangement of the masonry wall in which the double-leaf walls are well connected (case A). When the arrangement of

masonry is irregular or when the transversal connection between wall leaves is insufficient (Case B), the slenderness of the observed wall is doubled to take into account of the wall instability (Figure C. 8).

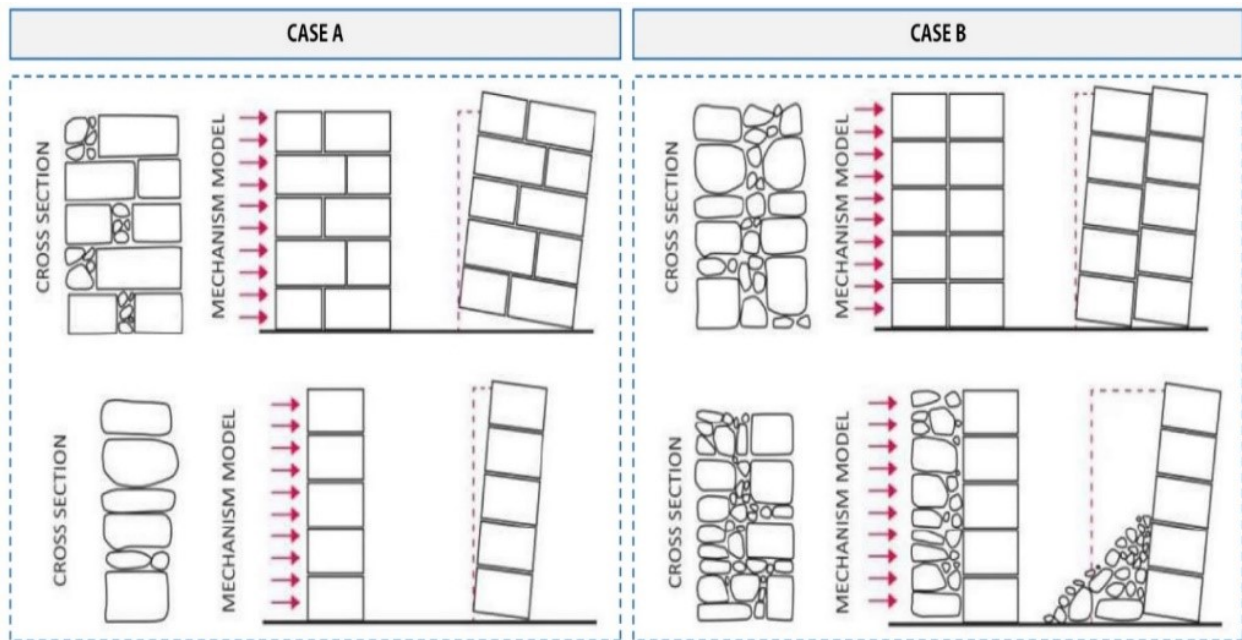


Figure C. 8: Structural behaviour of multiple-leaf stone masonry walls according to the quality of transversal connection among leaves (adapted from Ortega 2018).

The presence of a scale effect in rigid body motion, which makes the larger of two blocks with same slenderness ratio (H/b) more stable than the smaller one, is taken into account. It should be noticed that the presence of buttress attached to façade increases its base thickness, providing a low slenderness value. In this sense, the overturning mechanism can involve the portion of the façade above the buttress, rather than the entire façade. Figure C. 9 shows the activation of the partial overturning of the façade, a collapse mechanism that has been actually detected in the case study. The capacity of the system is evaluated through the analysis of the movement of the overturning point above the final portion of the buttress, considered on average at the base of the first floor for each façade configuration.



Figure C. 9: Activation of the partial overturning of the façade, a mechanism that is characterised by a horizontal crack in the final part of the buttress.

Table C. 11: Definition of the vulnerability classes for parameter P12

Slenderness (h/t)	Complete Overturning	Wall Composition	Parameter Class	Score
$h/t < 5$	No	Case A	B	5
		Case B	C	20
	Yes	Case A	A	0
		Case B	B	5
$5 \leq h/t < 7$	No	Case A	C	20
		Case B	D	50
	Yes	Case A	B	5
		Case B	C	20
$7 \leq h/t < 11$	No	Case A	C	20
		Case B	D	50
	Yes	Case A	C	20
		Case B	D	50
$11 \leq h/t$	No	Case A	D	50
		Case B	D	50
	Yes	Case A	D	50
		Case B	D	50

PARAMETER 8: Façade Openings

The façade openings influence the distribution of horizontal actions within SUs in aggregate: their dimension suggests the strength of the perimeter walls of the buildings, while their misalignment among the SUs indicates the presence of staggered floors. Parameter 13 exploits the experience acquired during the observation of the damage of the recent Italian earthquakes and the analyses performed by Vicente (2010) for the evaluation of the effect of the openings on the global seismic capacity of the SU. In this sense, the negative influence of the misalignment of the openings, both vertically and horizontally, is evaluated through the scheme suggested by the Vicente (2010) method and shown in Figure C. 10.

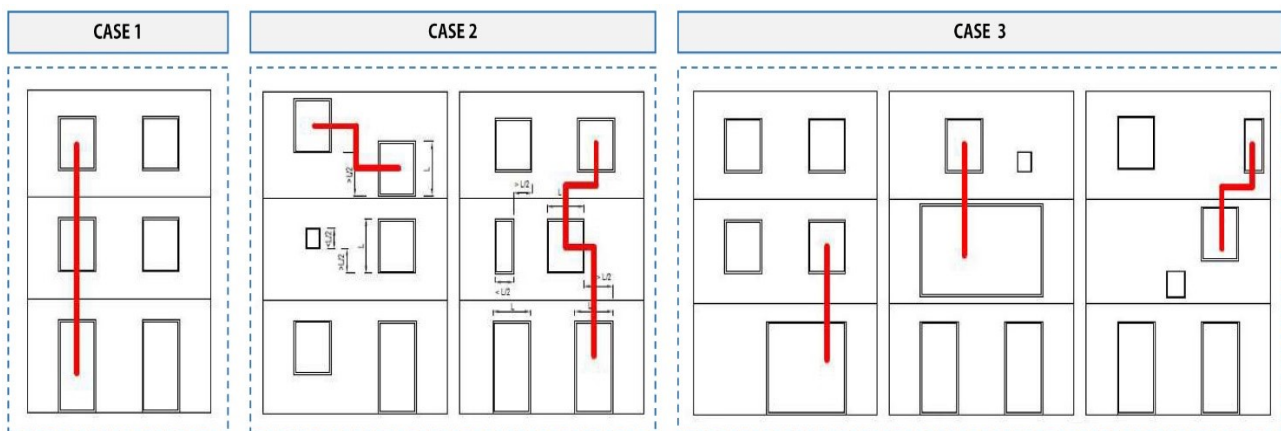


Figure C. 10: Schemes to identify the vulnerability classes, according to the possible configurations of the openings misalignments (adapted from Vicente 2010).

Finally, the value of the score increase according to the wall composition and openings misalignment: class A identifies façades with a percentage of openings lower than 5%, regular wall composition, well-connected masonry and regular or irregular openings disposition (i.e., vertically or horizontally misaligned more than $\frac{1}{2}$ of their width or $\frac{1}{2}$ of their height). The growth of the percentage of openings increases the vulnerability class, according to their misalignment and the wall composition.

Table C. 12: Definition of the vulnerability classes for parameter P13

Floor Openings	Wall composition	Openings Misalignment	Parameter Class	Score
Openings < 25%	Case A	Case 1	A	0
		Case 2	A	0
		Case 3	B	5
	Case B	Case 1	A	0
		Case 2	B	5
		Case 3	C	20
25% < Openings < 50%	Case A	Case 1	B	5
		Case 2	B	5
		Case 3	C	20
	Case B	Case 1	C	20
		Case 2	C	20
		Case 3	D	50
50% < Openings < 75%	Case A	Case 1	C	20
		Case 2	C	20
		Case 3	D	50
	Case B	Case 1	D	50
		Case 2	D	50
		Case 3	D	50

PARAMETER 9: The Façade Connections

The distribution of the metal tie-rods in the façade plays a significant role in ensuring the “Box Behaviour” of the structure, avoiding the activation of local mechanisms. In this sense, this parameter evaluates the effect of the wall connections in the out of plane behaviour of the building, considering the position and the number of ties in the façade. The damage detected in the recent Italian earthquakes reveals the positive effect of metal ties located on the upper floors of the building. Figure C. 11 shows how the vulnerability class should be assigned for the parameter “Façade Connections”, according to the buildings of the case study.

Table C. 13: Definition of the vulnerability classes for parameter P14

Position of ties in the SU	Number of Ties (x)	Parameter Class	Score
Top Floor	$1 < x$	A	-20
	1	B	-10
Middle Floors (not last neither first floor)	$1 < x$	B	-10
	1	C	-5
First Floor	$1 < x$	C	-5
	0	D	0



Figure C. 11: The number and distribution of metal ties, highlighted in red, leads to the assignment of class A (a), B (b) and C (c) for the buildings identified.

PARAMETER 10: The Corner Detachment

In general, the corner of masonry buildings is one of the most critical parts of the structure, as it defines the binding level between two orthogonal walls and, therefore, ensures the global behaviour of the building. In the case of the masonry aggregates, the corner is even more critical: the external SU is not confined by the adjacent SU, which provides a lateral constraint along one side of the building and therefore it is the most stressed building during a seismic event. The seismic damage show that the corner detachment is influenced by the slenderness of the corner walls, the dimension and position of the masonry wedge, the presence of ties on the two corners' sides, the presence of a diagonal roof beam at the top, the masonry quality and the regularity of the cornerstones (Table C.19). The presence of the metal tie at one floor doesn't prevent the activation of this mechanism on the upper floors. If the wall composition belongs to case B the parameter class assignment has to be reduced by one class (Figure C. 12).

Table C. 14: Structural features for the assignment of the vulnerability class

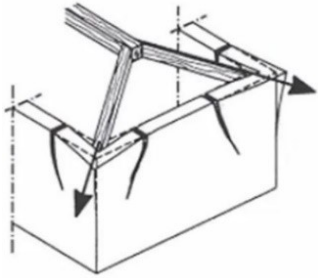
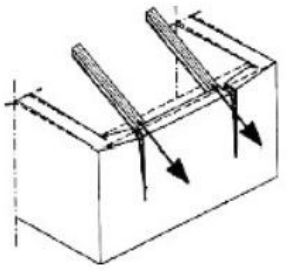
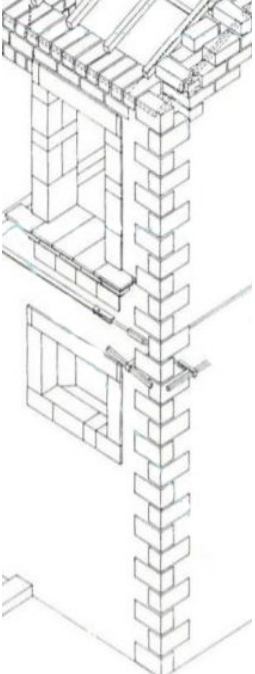
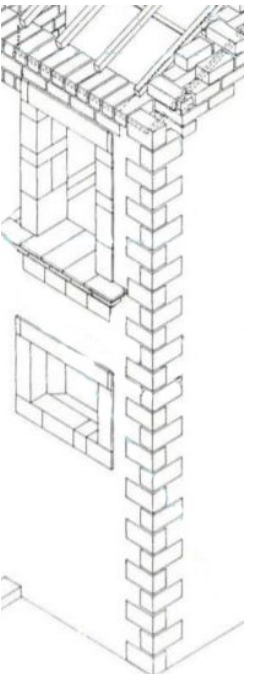
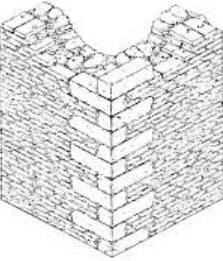
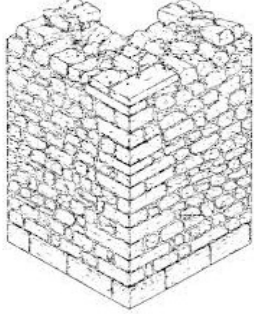
Presence of a Roof beam	Presence of metal ties	Masonry Quality
 Yes  No	 Yes  No	 Regular Cornerstone  No Cornerstone



Figure C. 12: Presence of tie rods at the corner of the aggregate (a); Corner failure of external buildings of an aggregate, caused by the absence of tie rods (b) and masonry connection between two orthogonal walls (c).

Table C. 15: Definition of the vulnerability classes for parameter P15

Wall Slenderness	Roof Beam	Metal Ties	Masonry Quality	Class	Score
$h/t < 5$	NO	YES	Regular Cornerstone	A	0
			No Cornerstone	A	0
		NO	Regular Cornerstone	A	0
			No Cornerstone	B	5
	YES	YES	Regular Cornerstone	B	5
			No Cornerstone	B	5
		NO	Regular Cornerstone	C	20
			No Cornerstone	C	20
$7 \leq h/t < 11$	NO	YES	Regular Cornerstone	B	5
			No Cornerstone	B	5
		NO	Regular Cornerstone	B	5
			No Cornerstone	C	20
	YES	YES	Regular Cornerstone	C	20
			No Cornerstone	C	20
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50
$11 \leq h/t$	NO	YES	Regular Cornerstone	C	20
			No Cornerstone	C	20
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50
	YES	YES	Regular Cornerstone	C	20
			No Cornerstone	D	50
		NO	Regular Cornerstone	D	50
			No Cornerstone	D	50

PARAMETER 11: Aggregate position and interaction

This parameter takes into account the position of the SUs within the SA and their mutual interaction, through the identification of staggered floors, which can generate, during an earthquake, thrusts that aren't balanced by the walls or the hammering effects. Four different positions are possible for the SU: isolated, at the top or in the end of an aggregate, inside of an aggregate and or at the corner with two free sides (Figure C. 13).

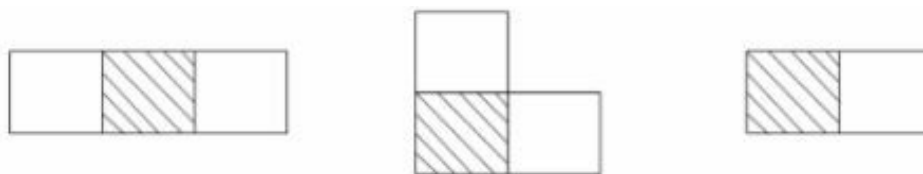


Figure C. 13: Possible positions of a structural unit within an aggregate

For each position of the SU a score is associated, which can be increased in relation of the number of staggered floors within the aggregate. In this sense, it is possible to take into account the presence of staggered floors if there is at least 50 cm of altitude of difference between them.

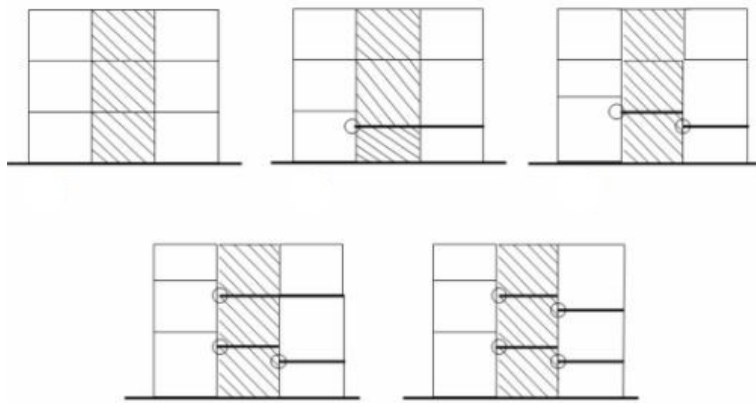


Figure C. 14: Possible positions of staggered floors

Table C. 16: Definition of the vulnerability classes for parameter P5

Position	Staggered floors	Class	Score
Between two buildings	Yes	A	0
	No	B	5
Isolated Building	Yes	B	5
	No	C	20
Corner Building	Yes	C	20
	No	D	50
Extreme Building	Yes	D	50
	No	D	50

PARAMETER 12: Plan regularity

The parameter analyses the planimetric shape of the building (Figure C. 15), by the calculation of two coefficients (β_1 and β_2) to define its class. In case of rectangular buildings, $\beta_1 = a/L$ defines the relationship between the shorter (a) and the longer side (L). In case of other regular plan form, it is necessary to take into account the measures of their deviations (b), with the variable $\beta_2 = b/L$.

The evaluation of the eccentricity between the center of mass and the center of stiffness is required in order to assess the plant irregularity. This procedure, more expensive and time consuming, can be applied after the geometrical survey of the building and allows to increase the vulnerability of the building.

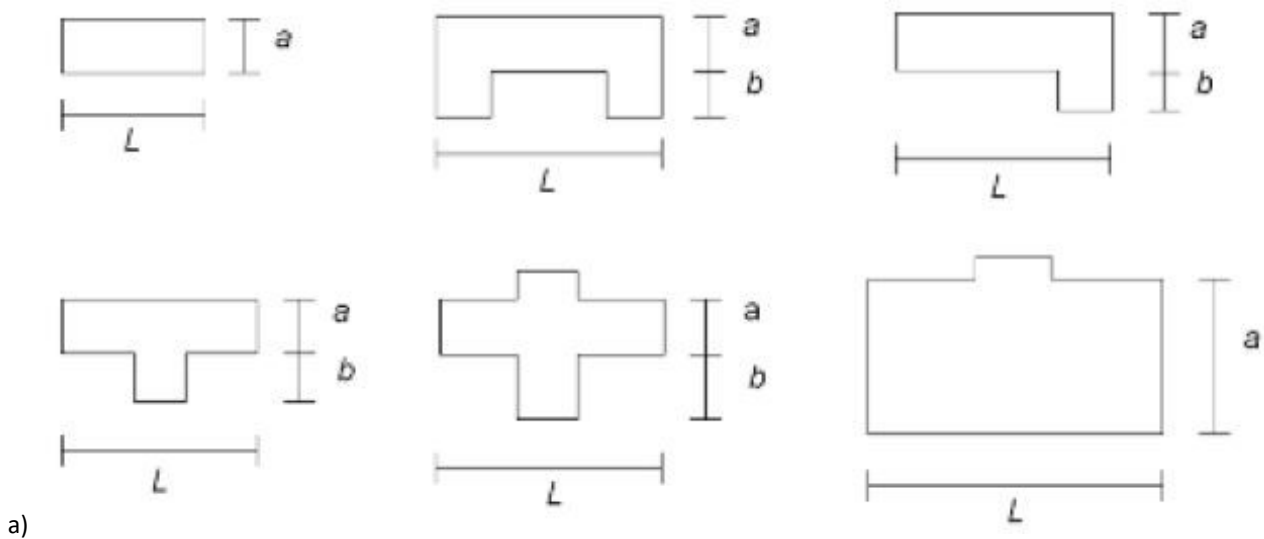


Figure C. 15 Scheme of the possible plan configurations.

Table C. 17: Definition of the vulnerability classes for parameter P6

Geometric Procedure		Eccentricity Procedure (space between the center of rigidity and the center of mass)	Class	Score
$\beta_1 \geq 0.75$	$\beta_2 \leq 0.1$	Less than 10% in comparison to the largest plant size	A	0
$0.5 \leq \beta_1 < 0.75$	$0.1 < \beta_2 \leq 0.2$	Between 10% and 20 % to the largest plant size	B	5
$0.25 \leq \beta_1 < 0.5$	$0.2 < \beta_2 \leq 0.3$	Between 20% and 30 % to the largest plant size	C	20
$\beta_1 < 0.25$	$\beta_2 > 0.3$	More than 10% in comparison to the largest plant size	D	50

PARAMETER 13: Height regularity

This parameter considers the presence of different stiffness at each level of the structure, considering the presence of porticos or lodges at the ground level or the existence of towers or turrets (Figure C. 16). The score is assigned by measuring the difference of masses at each level ($\pm \Delta M/M$), with a permissible deviation of 10% between consecutive floors. The ratio $\pm \Delta M/M$ can be replaced with the ratio $\Delta A/A$, which defines the covered area of the plan and its variation in different levels, considering the masonry type is the same at each level of the structure.

Another procedure is considered to assess the stiffness variation between floors, in order to indirectly define the height regularity. The deformation limits suggested by EC8 § 2.2.3 [CEN, 2004], given by the evaluation of the maximum inter storey drift due to the “frequent” earthquake, is used for the evaluation of stiffness distribution of the structure. The analysis of the non-linear behaviour of the SU, even with a simplified methodology, improves the evaluation of this parameter and therefore it is recommended but not mandatory

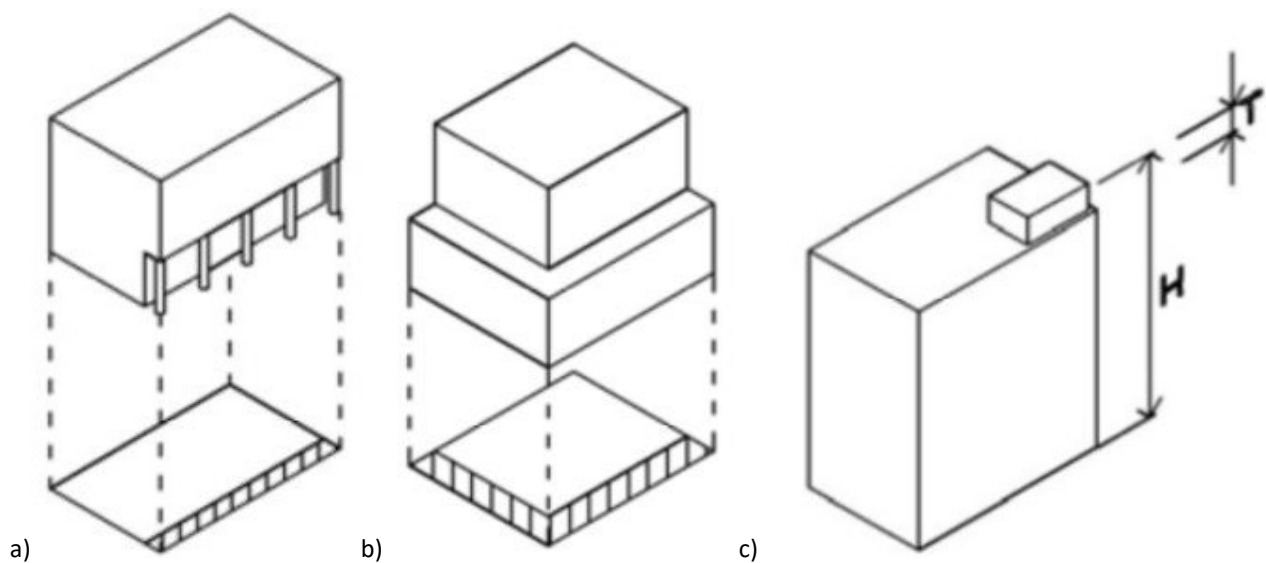


Figure C. 16: Scheme for the evaluation of the porticos dimensions (a), the plan difference of two consecutive floors (b), the presence of towers or turrets (c).

Table C. 18: Definition of the vulnerability classes for parameter P7

Simplified Procedure	Deformation limits (relative displacement divided by the interstorey height)	Class	Score
Building with constant mass distribution, as well as same floor area, along the height. Building with reduced mass or area in plan less than 10% ($\Delta M / M < 10\%$ or $\Delta A / A < 10\%$)	$\Delta \delta < 0.50\%$	A	0
Building with an area variation greater than 10% and less than 20% of the plant area ($10\% \leq \Delta M / M < 20\%$ or $10\% \leq \Delta A / A < 20\%$) Building with a tower less than 10% of the total height of the building ($T / H < 10\%$) Building with small gallery or arcade (corresponding to less than 10% of the total floor area)	$0.50\% \leq \Delta \delta < 0.75\%$	B	5
Building with an area variation greater than 20% of the area in plant ($20\% \leq \Delta M / M < 30\%$ or $20\% \leq \Delta A / A < 30\%$) Building with a tower of more than 10% and less than 40% total building height ($10\% \leq T / H < 40\%$) Building with gallery or arcade with a floor plan of more than 10% and less than or equal to 20% of the total floor area.	$0.75\% \leq \Delta \delta < 1.00\%$	C	20
Buildings with a retreat that represent a variation in area greater than 30% of the plant area ($\Delta M / M \geq 30\%$ or $\Delta A / A \geq 30\%$) Building with a tower of 40% or more in height total building ($T / H \geq 40\%$) Buildings with gallery or arcade with an area greater than 20% of the area total floor.	$1.00\% \leq \Delta \delta$	D	50

PARAMETER 14: Fragilities and conservation status

This parameter evaluates the overall conservation state of the building, looking at aspects like the crack pattern of the masonry walls (dimensions and width of the cracks), the out of plane of vertical structures and the decay of the observed structure (walls, floor, roofs). The vulnerability classes are identified through the magnitude of the criticisms detected that can trigger certain mechanisms or collapses following dynamic actions.

Table C. 19: Definition of the vulnerability classes for parameter P10

Description	Class	Score
Masonry is in good condition with no visible damage.	A	0
Masonry with capillary cracks (width < 0.5mm) scarcely spread. Presence of humidity that deteriorate the characteristics of masonry and lead to degradation of wood.	B	5
Masonry with modest cracks (width of the lesion: 2-3 mm) and characterized by a state of conservation of the walls, that can determine a significant decrease of resistance. High deformability problems of the stair structure, floor deformations, cracks in interior partition walls and in the corner of the openings	C	20
Masonry walls that show an out of plane behaviour, with a significant crack pattern and deterioration of materials' characteristics. Wide cracks in important places, such as near the corners (signs of disconnection between orthogonal walls). Damage caused by thrust covers, decay and degradation of the wood structural framework, cracking due foundation subsidence.	D	50

PARAMETER 15: Non-structural elements

This parameter takes into account the presence of other non-structural elements, like chimneys, additions, ceilings, etc. whose collapse can lead to damage, blockage or injured people. In this sense, the classes are divide in relation to their presence, their dimension and their connection to the vertical structures.

Table C. 20: Definition of the vulnerability classes for parameter P11

Description	Class	Score
Buildings without fixtures, additions or false ceilings.	A	0
Buildings with small chimneys, balconies and attached elements well connected.	B	5
Building with external elements, poorly connected to the main structure. Building with small roofs addition or ceilings poorly connected to the structure, or roofs of large extension well connected to the structure.	C	20
Building with chimneys and external elements with considerable weight (water deposits, additional roofs, mechanical equipment fixed to the walls), poorly connected to the structure. Building with balconies with poor connection to horizontal elements or built after the construction of the walls with poor connection. Building with a large and poorly connected roof or ceiling.	D	50

ANNEX D. Questionnaire

Dear colleague,

I would appreciate your feedback in an online survey.

As part of a PhD research project of the University of Pisa and University of Minho, we are developing a simplified methodology for the seismic vulnerability assessment of masonry buildings in historic city centres. The method is composed of 15 parameters, which account for the structural features that rule the global and local behaviour of masonry buildings in aggregate.

In the following survey, we ask you to compare the relative importance of the parameters, in relation to the seismic vulnerability, on a scale from 1 to 9 (where 1 means that two parameters have equal significance and 9 means that one factor has extremely higher importance than the other). The results will be processed using the Analytic Hierarchy Process Method (Saaty, 1987). Please refer to the following paper if you want to find out more about this methodology: <http://www.sciencedirect.com/science/article/pii/S0270025587904738>.

We prepared 50 questions, which we expect that it will take you no more than 10 minutes to complete. The following figure shows the construction typology for which this methodology is intended.

All responses will remain confidential and secure.

Thank you very much for your time and for sharing with us your valuable insight.

Images taken from the medieval town of Campi Alto di Norcia in Umbria (Italy), which was hit by the earthquakes of August 24th and October 30th, 2016.



1/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc)
- ☐ NUMBER OF FLOORS: Average height of the building
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

2/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc)
- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:*Mark only one square*

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

3/50**Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:***Mark only one square*

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:*Mark only one square*

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

4/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

5/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

6/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:*Mark only one square*

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

7/50**Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:***Mark only one square*

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ HEIGHT REGULARITY: Distribution of mass in height.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:*Mark only one square*

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

8/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ QUALITY OF RESISTING SYSTEM: Define the quality of the connection between the piers (presence of ties, concrete rings, etc).
- ☐ FRAGILITIES AND CONSERVATION STATUS: Crack pattern of the structure.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

9/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ NUMBER OF FLOORS: Average height of the building.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

10/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

11/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

12/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

13/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

14/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ PLAN REGULARITY: Disposition of the external walls in plan.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

15/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CONVENTIONAL STRENGTH: Base shear resistance of the building.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

16/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building
- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

17/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building
- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

18/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building
- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

19/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building
- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

20/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building.
- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

21/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building.
- ☐ HEIGHT REGULARITY: Distribution of mass in height.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

22/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ NUMBER OF FLOORS: Average height of the building.
- ☐ FRAGILITIES AND CONSERVATION STATE: Crack pattern of the structure.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

23/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

24/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

25/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

26/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

27/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FLOOR TYPOLOGY: Rigid or deformable horizontal diaphragms.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

28/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

29/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

30/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

31/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ HEIGHT REGULARITY: Distribution of mass in height.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
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Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other
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Intensity of importance	Definition	Explanation
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

32/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ ROOF TYPOLOGY: Concrete or timber roof and thrust influence.
- ☐ FRAGILITIES AND CONSERVATION STATE: Crack pattern of the structure.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

33/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

34/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

35/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

36/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ LOCATION OF THE BUILDING AND SOIL LOCATION: Morphology of the ground surface and foundation typology.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

37/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
3	Moderate importance of one over another	Experience and judgment slightly favor one parameter over another
5	Essential or strong importance	Experience and judgment strongly favor one parameter over another
7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

38/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

39/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ PLAN REGULARITY: Disposition of the external walls in plan.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

40/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE SLENDERNESS: Ratio between the height of the facade and thickness at the base.
- ☐ FRAGILITIES AND CONSERVATION STATE: Crack pattern of the structure.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

41/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor..
- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
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Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other
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Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

42/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor..
- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
1	Equal importance	Two parameters contribute equally to the objective
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

43/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor..
- ☐ HEIGHT REGULARITY: Distribution of mass in height.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

44/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE OPENINGS: Ratio between the openings' area and the total area of the wall, measured at each floor.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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7	Very strong importance	A parameter is strongly favored and its dominance demonstrated in practice
9	Extreme importance	The evidence favoring one parameter over another is of the highest possible order of affirmation
2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

45/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

46/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FACADE CONNECTIONS: Presence of metal ties at each floor.
- ☐ PLAN REGULARITY: Disposition of the external walls in plan.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

47/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ CORNER DETACHMENT: Quality and regularity of the corner stones, presence of ties and roof beams at the top.
- ☐ FRAGILITIES AND CONSERVATION STATE: Crack pattern of the structure.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

48/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ AGGREGATE POSITION AND INTERACTION: Position of the building in the aggregate and floors misalignment.
- ☐ HEIGHT REGULARITY: Distribution of mass in height.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

49/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ PLAN REGULARITY: Disposition of the external walls in plan.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

Intensity of importance	Definition	Explanation
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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

50/50

Please, indicate which of the following parameters you consider has a greater influence on the seismic behaviour of masonry buildings in aggregate:

Mark only one square

- ☐ FRAGILITIES AND CONSERVATION STATE: Crack pattern of the structure.
- ☐ NON - STRUCTURAL ELEMENTS: Chimneys, tiles, balconies and all non-structural elements connected to the facade.
- ☐ Equal Importance

Compare their relative importance in the provided scale from 1-9:

Mark only one square

	1	2	3	4	5	6	7	8	9	
Equal importance	☐	☐	☐	☐	☐	☐	☐	☐	☐	Extreme importance of one parameter over the other

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2,4,6,8	Intermediate values between two adjacent judgments	When compromise is needed

Personal Details

To conclude, we would like to ask you for some personal details that will help us to weigh the survey, thank you.

Name

Professional Position

Mark only one square

- ☐ Professional
- ☐ University Professor
- ☐ Postdoc Researcher
- ☐ PhD Researcher
- ☐ Student
- ☐ Other

How would you rate your overall level of expertise on the matter in question on a scale of one to five?

Mark only one square

	1	2	3	4	5	
Low expertise	☐	☐	☐	☐	☐	High expertise

Send me a copy of my responses.

