وزارة التعليم العالي و البحث العلمي

BADJI MOKHTAR UNIVERSITY- ANNABA
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FACULTE DES SCIENCES DE L'INGENIORAT
DEPARTEMENT DE GENIE CIVIL

Année : 2015

THESE

Présentée en vue de l'obtention du diplôme de DOCTORAT en Génie Civil

Thème

Seismic vulnerability and risk assessment of Annaba region (Algeria)

Option : Structure

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ACKNOWLEDGEMENTS

In the name of Allah, the Most Gracious and the Most Merciful Alhamdulillah, all praises to Allah for the strengths and His blessing in completing this Phd thesis.

During the period of my research in the interesting field of the seismic risk at Annaba city, many people have contributed in various ways to this final dissertation. Firstly, I would like to thank my supervisor Dr. Gouasmia Abdelhacine for his aid and encouragements constantly provided during the period of my Phd.

A special thanks to Dr. Boukri Mehdi; a researcher at the Algerian National Earthquake Engineering Centre (CGS) for his guidance and learning me the basic elements of the applied methods.

I thank also the Annaba city council (technical organism in charge of the Technical Control of Construction, CTC) and DUCH (Direction of Urban planning and Construction of Habitation) for providing the data of the study area.

I wish to express my deep acknowledgment to Professor Romeu da Silva Vicente for his help and the opportunity he gave me to do a research training period in Aveiro University - Portugal and the guidance during my stay together with my best friend Dr. Tiago Miguel Ferreira. Sincerely, without their support and advice, this work would not be possible.

In this thesis, very valuable information about the historical seismicity and the active faults located in Algeria are presented as well as the probabilistic seismic hazard assessment of Annaba city. This fact leads me to be very grateful to Dr. Mezouar Nourredine; also a researcher at the Algerian National Earthquake Engineering Centre (CGS) for the data provision and Dr. Assia Harbi, research director of CRAAG; the Algerian Centre for Research in Astronomy, Astrophysics and Geophysics in addition for her help in the very difficult moments.

My deepest gratitude goes to my sister and brothers, to my wife Khemis Asma; Phd student at the same Department of Civil Engineering - University of Badji Mokhtar Annaba -, and her family for their love and invaluable care.

Finally, in fact that I did not find a way to thank the dearest two persons in my heart, I have to leave my God to thank them instead of me. They are my parents, who have always given me the strength and wisdom to be sincere in my work, for setting high moral standards and supporting to reach my dreams, and for their unselfish love and affection. This dissertation is dedicated to them.

Abstract:

This thesis addresses and discusses the seismic risk associated to the old urban areas of the Algerian city of Annaba. In this sense, two seismic vulnerability index methods and damage estimation have been adapted and applied wherein the first one is based on the EMS-98 building typologies, and the second is based on the Italian GNDT approach. To accomplish this task, we employed an existing data survey, which, however, was not originally developed for seismic purposes. It was used herein to provide input to the vulnerability methods. The goal of this research thesis was not only to assess the seismic vulnerability and expected damage within Annaba, but also to adapt the two mentioned approaches to the existing data survey, aiming to use such nonspecific building database and to study the possibility of its application for seismic risk estimation in similar regions. About 380 of historical masonry buildings were assessed using the selected methodologies, aiming at evaluating the expected physical damage.

Subsequently different seismic scenarios were developed at urban-scale analysis, in order to evaluate the relevant expected economic and human losses in the studied area. For depicting and analyzing the obtained results from the application of the two methods, the results were integrated and compared using a geographic information system (GIS), which proving that the vulnerability of the buildings surveyed in Annaba is significant and therefore public awareness are required. Consequently, the ultimate goal is to support the city council of Annaba for the implementation of risk mitigation and emergency planning strategies.

RESUME

Cette thèse, adresse et discute le risque sismique associé aux anciennes zones urbaines de la ville Algérienne d'Annaba. À cet égard, deux méthodes d'indice de vulnérabilité sismique et estimation des dommages ont été adaptées et appliquées, dont la première est basée sur les typologies de construction EMS-98, et la deuxième est inspirée de l'approche Italienne GNDT. Pour accomplir cette tâche, nous avons utilisé une enquête de données existante, qui, cependant, n'a pas été développée à l'origine à des fins sismiques. Elle a été utilisée ici pour apporter une entrée de données aux méthodes de vulnérabilité. L'objectif de cette thèse de recherche n'était pas seulement d'évaluer la vulnérabilité sismique et les dommages attendus à Annaba, mais aussi d'adapter les deux approches mentionnées à l'enquête de données existante, dans le but d'utiliser cette base de données spécifique des constructions et d'étudier la possibilité d'évaluer le risque sismique dans des régions similaires. A propos de 380 bâtiments historiques en maçonnerie ont été évalués en utilisant les méthodologies choisies, visant à évaluer les dommages physiques attendus. Par la suite, différents scénarios sismiques ont été développés à l'analyse d'échelle urbaine, afin d'évaluer les pertes économiques et humaines escomptés pertinents dans la zone étudiée. Pour illustrer et analyser les résultats obtenus à partir de l'application des deux méthodes, les résultats ont été intégrées et comparées en utilisant un système d'information géographique (SIG), qui a prouvé que la vulnérabilité des bâtiments étudiés d'Annaba est importante, et donc la sensibilisation du public est nécessaire. Par conséquent, le but ultime est de soutenir le conseil de la ville d'Annaba pour la mise en œuvre des stratégies d'atténuation des risques et de planification d'urgence.

ملخص

هذه الأطروحة تناقش مخاطر الزلازل المرتبطة بالمناطق الحضرية القديمة في المدينة الجزائرية عنابة. وفي هذا الصدد، تم تكييف طريقتين مؤشر الضعف الزلزالي وتقدير الأضرار وتطبيقها حيث تقوم الأولى على أنماط بناء EMS-98، وتستند الثانية على نهج GNDT الإيطالي. لإنجاز هذه المهمة، وظف الكتاب مسح بيانات موجودة التي، مع ذلك، لم تكن وضعت أصلا لأغراض زلزالية. كانت تستخدم هنا لتوفير مدخلات لطريقتين الضعف. الهدف من هذه الأطروحة البحثية ليس فقط لتقييم الضعف الزلزالي والضرر المتوقع في عنابة، ولكن أيضا تكيف اثنين من النهج المذكور على الدراسة الاستقصائية البيانات الموجودة، وتهدف إلى استخدام هذه قاعدة البيانات غير محددة والمبنى على دراسة إمكانية من تطبيق للحصول على تقدير مخاطر الزلازل في مناطق مماثلة. حوالي 380 من المباني التاريخية حجرية تم تقييمها باستخدام المنهجيات محددة، تهدف إلى تقييم الأضرار المادية المتوقعة. بعد ذلك تم تطوير سيناريوهات زلزالية مختلفة في تحليل على نطاق المناطق الحضرية، بهدف تقييم الخسائر الاقتصادية والبشرية المتوقعة ذات الصلة في منطقة الدراسة. لتصور ومقارنة وتحليل النتائج التي تم الحصول عليها من تطبيق الطريقتين تم استخدام نظام المعلومات الجغرافية (GIS)، والذي أثبت أن ضعف المباني التي شملتها الدراسة في عنابة كبير، وبالتالي هناك حاجة الوعي العام. وبناء على ذلك، فإن الهدف النهائي هو دعم مجلس مدينة عنابة لتنفيذ استراتيجيات التخفيف من المخاطر والتخطيط لحالات الطوارئ.

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Chapter I: Seismic risk assessment

I.1 Generality

I.1.1 Introduction

The seismic phenomenon has lived with the humans since the start of the times. Man, throughout history has always encountered with the earthquake as a natural disaster, and has undergone social and economic losses as a result of it. Generally the lack of man knowledge and his inability to intervene in natural environments, such as excessive construction in faults vicinity, as well as inattention to build with regulations and standards, turn earthquake to a disaster phenomenon. Nowadays, these and many other factors have made sever the threat of accidents resulting from natural phenomena, especially earthquake, and as a result of its occurrence there will be great crisis in human society. In this context, assessing the impact of an earthquake upon an urban area is important for many reasons, one of them is that decision makers can develop disaster impact scenarios, which can be subsequently used to outline preventive measures, applicable to mitigate the probable unlike consequences of the seismic event. Consequently, bolstering the resiliency of communities to such hazardous events should begin with a comprehensive risk assessment. The earthquake risk assessment is a comparably young discipline defined by several researchers namely by , Cardona (2001), Coburn and Spence (2002), McGuire (2004), Barbat *et al.* (2010) or Maio *et al.* (2015) as a mathematical convolution between hazard, vulnerability and exposure (Eq. I.1):

$$\text{Risk} = \text{Hazard} \otimes \text{Vulnerability} \otimes \text{Exposure} \tag{I.1}$$

Where the each parameter is defined as following:

1. **Hazard:** refers to a latent danger or an external risk factor of an exposed element that can be expressed as the probability of occurrence of an event of certain intensity in a specific site (Barbat *et al.* 2010). In this sense, the seismic phenomena can be defined as the likelihood of occurrence of a potential damaging earthquake within a given time frame, characterized for being an inevitable event (out of human control) in the area of its location.
2. **Vulnerability:** Seismic vulnerability is an intrinsic property of the structure, a characteristic of its own behavior due to the occurrence of an earthquake of certain intensity described trough a law of cause-effect, where the cause is the seismic action and the effect is the amount of damage that could be presented in a building. According to Vicente *et al.* (2011), it can be used to indirectly measure the reduction in a building's structural efficiency or a building's residual ability to guarantee its expected use and function under normal conditions.
3. **Exposure:** the exposure reflects the value of the exposed elements at risk (Ferreira *et al.* 2013) which corresponds to the conjunct of the potential social, economic and cultural consequences in the built environment and persons due to earthquakes.

In the present thesis, the use of the term “seismic risk” represents a true inter-disciplinary research field since it requires the expertise and knowledge of a number of research areas (Fig. I.1) (Lang 2012) such as:

- seismology, or more precise seismic hazard assessment (SHA),
- geology and tectonics,
- geotechnical and structural (earthquake) engineering,
- urban land-use planning
- sociology, or more precise disaster sociology,
- insurance/re-insurance industry,
- disaster management and emergency relief, as well as
- Geographical Information Systems (GIS).

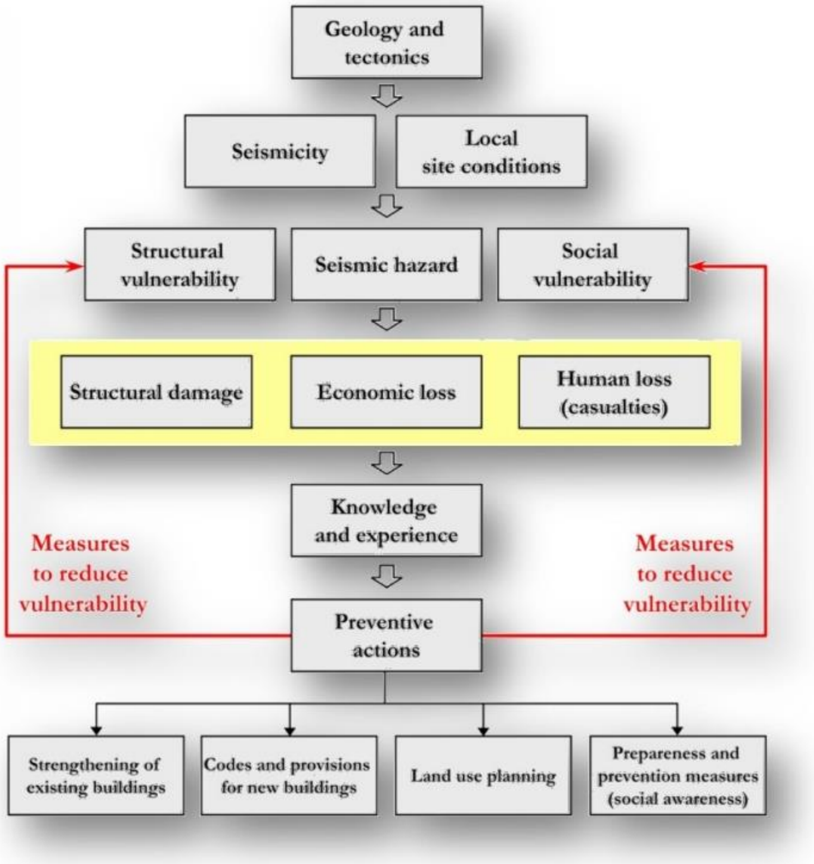


Fig. I.1: The causal chain from basic research disciplines to preventive actions through earthquake loss estimation (Lang 2012).

Considering the above flow chart (Fig. I.1), it can be concluded that the main purpose of studying the seismic risk upon a buildings stock, is to generate reliable assessment of expected physical damage as well as the economic and social losses that are connected to the damages either in a direct or indirect way. The amount of damage identified in the seismic vulnerability assessment of buildings depends on many factors such as intensity of the seismic action, soil conditions, constructive materials, state of previous damages and structural elements. Another important aspect to consider is whether the structure was designed to resist earthquakes or only to withstand its own self-weight, such as the case of the historical masonry constructions. In this context, considering such typology of buildings stock which is the one presenting the greatest vulnerability (Grünthal 1998), the dilemma of their seismic risk assessment is herein studied especially for the case of the urban areas of Annaba city.

I.1.2 Management and tools

A risk management involves specific planning, practices of mitigation before earthquakes and provision of critical and timely information to improve response of the society after the calamitous event. Regarding the urban areas, the risk management in many cases is undertaken without the use of a general planning tool. A primary consequence of this situation is that city councils or regional authorities (decision makers) do not acquire a global view of the area under analysis, which can compromise the effectiveness of their rehabilitation strategies and risk mitigation measures (Vicente *et al.* 2008). This fact justifies the need for a developed tool connected to a relational database, through which it is possible to perform integrated analysis of the studied area.

In this context, the GIS (Geographic Information System) represents, nowadays a suitable instrument for multi-disciplinary studies and for the scenario or risk analysis performance. Indeed, throughout a GIS employment, various and complex aspects could be controlled at different scale to develop a complete seismic risk analysis by means of the interaction of hazard estimation, exposure identification and vulnerability assessment.

The GIS system permits to cross different kind of data and verify, from many points of view, the effects deriving from specific territorial phenomena. In particular, a GIS allows to computerize capture, store, analyze, manage and show data linked to a specific zone by means the use of the personal computer. The GIS application software combines geo-referenced graphical data (vectorised information and orthophoto-maps) with building's information. In this specific case, each polygon (represent a building) was associated with several features and attributes, allowing for their visualization, selection and search (Ferreira *et al.* 2013). Two types of spatial views were possible: a global view of the entire area under study and, alternatively, a local view of each defined zones.

All calculations of the likelihood multiple damage (such as number of collapsed and unusable buildings) and loss scenarios (like the rate of human death and injuries, repair costs, etc.) presented in this thesis work are programmed in Excel file for different earthquake intensities. This procedure enables rapid data editing of building information, depicting high quality of the scenarios outcomes, which in turn support risk management actions and decisions.

All database information associated with GIS application could be updated at any time especially from the prepared Excel file, that's why this kind of treatment is very rapid and useful especially for the management of the building stock under consideration.

In this work, the potentialities of GIS have been extensively proved in implementing of a mutable and progressive platform that integrated all aspects of seismic risk evaluation of a historical center, from building characteristics to the estimation of economic loss as it will be shown in next Chapters.

I.2 Outline of the thesis

I.2.1 Contribution

The main work of this thesis is studying the seismic risk upon an urban district of the city of Annaba through the creation of damage and loss scenarios. Such a study is carried out by mean of comparison between results derived from different urban vulnerability methods using an existing building survey previously performed for other purposes. To perform this task, implicit assumptions and explicit modifications in order to minimize the uncertainties of the final results are applied.

Various methods have been implemented in different countries to assess the seismic vulnerability of buildings wherein their application have shown that among all, those methods based on vulnerability indexes are more suitable for large-scale assessments. Despite their high level of approximation and uncertainties, they present the advantage to consider a large number of buildings (Quiroz et al. 2010) for a first estimate at the city or urban district scale when only limited and general data are available.

The establishment of a new, or the adaptation of an existing, vulnerability assessment approach should take into account criteria suitable to the specific characteristics of the area under study. In particular, the seismic hazard of the site together with the vulnerability of the building tissue depending on the structural architecture, the construction material, the character of use, and the state of health/preservation/robustness of the buildings (Ferreira *et al.* 2013, Ferreira *et al.* 2014). It is worth mentioning that when developing an urban earthquake scenario, more detailed inventory and classification scheme should be considered, which at times can be originated from different purposes. In this regard, the building data used in this work comes from an existing general program of a building-by-building detailed survey performed by the CTC (the official technical organization of Annaba city in charge of the Technical Control of Construction), wherein the buildings were classified according to their degradation state, towards setting retrofitting and maintenance priorities (CTC 2010).

Considering the aims of this thesis, two seismic vulnerability index methods, specifically developed for the Euro-Mediterranean area, were selected among the available methodologies; the first one designated as RISK-UE LM1 method (Mouroux and Brun 2006, RISK-UE 2003), which is based on the EMS-98 building typologies (Grünthal 1998), and the second one based on the Italian GNDT approach level II (Benedetti *et al.* 1988, GNDT 1993b). The main differences found between the results obtained from each of the two methods are compared and discussed in this thesis, taking into account the context of Annaba city and in view of their replication on future seismic vulnerability assessment works apply to other old urban areas in Algeria.

Subsequently, an equivalence between both selected methods is then performed in the aim of assessing for the different seismic scenarios previously evaluated, the likelihood physical losses (rate of buildings in collapse and those unusable) the expected relevant economical and human losses, respectively in terms of repair cost in Dinar of Algeria and the rate of death and injuries people for the built-up area of the old town of Annaba city based especially on the modified GNDT II approach which is considered herein as the main applied method.

Finally, it is important to underline that the outputs obtained are then mapped using a GIS tool connected to a relational database, enabling the storage and the spatial analysis of the results into an open, georeferenced and fully upgradable environment.

I.2.2 Motivation

Extensive damage was observed following earthquakes in urban built environments that are concerned especially by the historical masonry structures. The first motivation that attract us to perform this research work, in one hand is that the experience shows a high vulnerability of such buildings to seismic actions, even of moderate intensity, wherein after their collapse, a great amount of people that are killed or seriously injured. In the other hand, the sum of all people that become homeless generates not only an economical but a big social problem difficult to deal with.

In this context, the second motivation is presenting in the case of Annaba (one of the northeast Algerian cities) which is assumed to have a moderate seismic hazard (Aoudia *et al.* 2000, Boughacha *et al.* 2004, Hamdache 1998, Harbi 2006, Kherroubi *et al.* 2009, Peláez *et al.* 2005). However according to the recent probabilistic microzonation study performed by the Algerian National Earthquake Engineering Centre (CGS 2011) concludes that hazard is significantly higher, with peak ground acceleration (PGA) levels ranging from 0.18 to 0.32g for 200–500year return periods respectively. Therefore, albeit disputed, such considerable hazard level of Annaba necessitates addressing the issue of the seismic risk analysis especially in the case of the historical masonry buildings located in its old city center, due to their accelerated degradation state, material heterogeneity, and past intrusive structural and non-structural modifications. These facts suggest on focusing research efforts to develop tools that emphasizes in these particular conditions. In this regards, to estimate the seismic risk of such building stock, the use of a reliable but expedite assessment method able to accounting the buildings features that most influence their seismic vulnerability is an essential part of a meaningful seismic risk analysis. However, in the case of Annaba city, this task leads to non-accurate and unreliable quantitative building vulnerability, due to the lack of information related to past seismic activities and losses, which are typically used for the calibration and validation process. In this case, the application or the adaptation of existing vulnerability methods already validated in the area of the Euro-Mediterranean region that Algeria belong to, is advisable. The main motivation behind this assumption is founded on the thesis that, accounting for the structural characteristics and urban organizations of the Algerian buildings, they can be considered similar to those which have been studied in Europe and Algeria (Senouci *et al.* 2013). This assumption is further supported by the great compatibility found between the existing information of the study area (CTC data) and the main parameters used in both methods (Athmani *et al.* 2014). Therefore, as already mentioned, two European approaches are chosen to be adapted and applied in the context of the built-up environment of Annaba city.

At the end, it is absolutely essential to understand the seismic risk associated to those built environments in a reliable and accurate way. Indeed, knowing the seismic risk and potential losses allows for proper budgetary planning, raising public awareness, assessment and allocation of the necessary manpower for mitigation and disaster management operations, prioritization of retrofit applications, and educating the public and professionals on preparedness.

I.2.3 Organization

According to what exposed above, this thesis is structured in six chapters. The Chapter I presents the generality on the seismic risk assessment highlighting our contribution, and shortly describes the motivation for choosing this research topic as well as provides a general overview of the objectives of the thesis.

The Chapter II, Chapter III and the Chapter IV are related respectively to one parameter that contributing in the definition of the meaningful seismic risk as described in Eq. (I.1). In this context, in descending way starting from the north of Algeria to the its north-east reaching to the our area of interest of Annaba city, the Chapter II consists of identifying and characterizing the active faults and all earthquake events occurred in these zones emphasizing those of significant impact, especially on the old building stock of Annaba.

With regard to vulnerability analysis which is the main element accounting in Chapter III, a review of the existing and most applied vulnerability methods is presented with the aim to accentuate some limitations to be overcome rather than to provide an exhaustive state of the art. In this Chapter, the selected vulnerability approaches are presented in details in their original version. The first one is the Italian GNDT approach level II which is so-called “mother method”, and the second one designated as RISK-UE LM1 method.

Subsequently, the Chapter IV, was reserved for the presentation of the exposure and the elements at risk located in the area of our interest of Annaba city. Additionally, the available data of the buildings stock provided from a general program of a building-by-building detailed survey performed by the CTC (the official technical organization of Annaba city in charge of the Technical Control of Construction) are exhibited.

In the Chapter V, new adaptation of the selected methodologies (GNDT II and RISK-UE) is carried out with which the earthquake risk posed to the Annaba city is quantified by considering the vulnerability of its building stock in in a short, medium and a long-term of seismic hazard. In this regard, the physical damages for the studied historical buildings of the old city center of Annaba for different seismic scenario is presented, wherein the necessary vulnerability curves are drown. Accordingly, the expected relevant losses in terms of collapse and unusable buildings as well as in terms of homeless and casualty people are evaluated for each seismic scenarios. The expected economic losses in term of repair cost of the entire building stock under study demanded in each seismic scenario is also estimated in term of Algerian Dinar (DA).

Finally, the last part of the thesis is reserved for the appendix, where the vast set of building data used as input in the previous chapter to feed the applied methodologies are manipulated. Therefore, based on the description of the inherent parameters defined in each original version of the applied methodologies, their possible application using the non-ad hoc data survey of CTC was carried out in detail. Moreover, the use of the new additional parameters inserted in each method according to the same data source (CTC) is also presented. At the end, the spatial and the frequency distribution of each parameter are depicted for all masonry buildings of the study area.

Chapter II: Seismic hazard

II.1 Introduction

The term of hazard is described in the previous studies with different ways however leads to the same meaning. For instance, “a hazard, in a broader definition, is a threat to people and the valuable things. In other way, “Hazard … reflects a potential threat to humans as well as the impact of an event on society and the environment. Therefore, Hazards arise from the interaction between social, technological, and natural systems” (Katharina 2006).

In engineering terms, hazard was described as an extreme geophysical event that is capable of causing a disaster wherein its fundamental determinants are location, timing, magnitude and frequency (Katharina 2006). In this regards, natural hazard is used in a mathematical sense to mean the probability of the occurrence, within a specified period of time and a given area, of a potentially damaging phenomenon of a given intensity (Katharina 2006).

Focusing on the natural phenomena of earthquake, the seismic hazard analysis consists on the estimation and description of the ground motion by an appropriate parameters, which are represented under suitable form of maps required by a seismic risk analysis. This kind of hazard assessment involves studies of historical data, skilled interpretations of existing topographical and geological maps (Gilda 2010).

II.2 Evaluation of the seismic hazard

The goal of many earthquake engineering analyses is to ensure that a structure can withstand a given level of ground shaking while maintaining a desired level of performance. But what level of ground shaking should be used to perform this analysis ?. Given the challenges in earthquake prediction, seismic hazard analysis is considered one of the practical solutions to cope with the complicated, random earthquake process (Wang and Huang 2014). First of all, it should be worth clarifying that such an analysis is not to estimate the damage (casualties and economic loss) caused by earthquakes but to best estimate the level of earthquake-induced ground motion at the site. The seismic hazard characterization of a certain zone under study has been accepted as an engineering solution to the uncertain earthquake process (Wang and Huang 2014) which is recommended to be estimated by considering a combination of seismological, geophysical, geological and geotechnical studies with the history of earthquakes, damages and the experts’ opinion. Site-specific seismic hazards evaluated are then used for developing site-specific earthquake resistant designs.

Two approaches, Deterministic Seismic Hazard Assessment (DSHA) and Probabilistic Seismic Hazard Assessment (PSHA), are commonly used for evaluating seismic hazard. Implicitly, DSHA adopts ‘‘deterministic’’ information during analysis (Mualchin 2005, Wang *et al.* 2012), and PSHA accounts for the ‘‘probabilistic’’ characteristics of earthquake size, location, and ground motion models (Cornell 1968). In broad definition, the deterministic considers each seismic source separately and determines the occurrence of an earthquake of specified size at a specified location. The probabilistic combines the contributions of all relevant sources and allows characterizing the rate at which earthquakes and particular levels of ground motions occur (Giovinazzi 2005). A few case studies have been reported with the two methods (Atkinson and Goda 2013, Azzaro *et al.* 2013, Bommer 2002, Bozzoni *et al.* 2011, Moratto *et al.* 2007, Ordaz *et al.* 2014, Wang and Huang 2014).

It is worth to note that the choice of the parameter to be employed for the ground motion characterization depends definitely on the quality of the analysis to be performed. Additionally, the selection of the most suitable parameter for the ground motion description must be coherent with the vulnerability model chosen for the seismic building behavior assessment, for instance; the employment of a physical-mechanical parameter could be for instance inappropriate if reference is made to an observational vulnerability model (Giovinazzi 2005).

II.2.1 Deterministic seismic hazard analysis (DSHA)

The Deterministic Seismic Hazard Analysis has been the earliest approach taken for seismic hazard analysis. This approach (DSHA) uses the geology and seismic history data to identify earthquake sources (single faults or fault zones) to define separately the earthquake scenarios by magnitude, distance between source and area, style of faulting and in some cases rupture direction (Gürboğa and Sarp 2013). The DSHA is proposed on the condition of choosing the worst-case earthquake scenario, usually related to the Maximum Credible Earthquakes (MCEs); the largest earthquakes that can reasonably be expected. The ground motion for the scenario earthquake is usually estimated by using attenuation relationship. These latter take into account the effects of earthquake waves traveling from the epicenter to the site on the ground motions. Local site conditions also need to be considered (Ade *et al.* 2011). Therefore, DSHA results in an estimation of the ground motion at the site of interest due to the specified scenario earthquake. A typical DSHA can be described in four-step process (Fig. II.1, (Gürboğa and Sarp 2013):

1. Identification and characterization of all sources capable of producing significant ground motion at the site. Providing an exhaustive documentation of the seismic history of the selected site, related to the geological faults, the magnitude of the maximum historical earthquakes and their distance from the site is required. The information can be acquired referring to specific data source, such as earthquake catalogues or seismo-tectonic studies. In general, the territory is divided with cells in order to define a grid. The center of each cell is constituted by a seismo-genetic source, where the maximum magnitude observed in the epicenter area and the distance from the studied region are fixed.

2. Selection of a source-to-site distance parameter for each source zone, generally assumed in order to represent the most unfavorable situation, where the shortest distance between the point or source zone and the site of interest is selected. The distance may be expressed either as an epicentral or hypocentral distance, depending on the measure of distance of the predictive relationship used in the following step.

3. Comparing the level of shaking produced by earthquakes (identified in step 1) assumed to occur at the distances identified in step 2. A controlling earthquake is selected (i.e, the earthquake that is expected to produce the strongest level of shaking) calculating by using an attenuation relationship enabling to estimate the ground shaking within the area of

interest, which is described in terms of its size (usually expressed as magnitude) and distance from the site.

4. Finally, the hazard can be defined as the maximum expected value, usually described by one or more ground motion parameters obtained from predictive relationships. Peak ground acceleration (PGA), peak velocity (PGV) and response spectrum ordinates (S_a) are commonly used to characterize the seismic hazard.

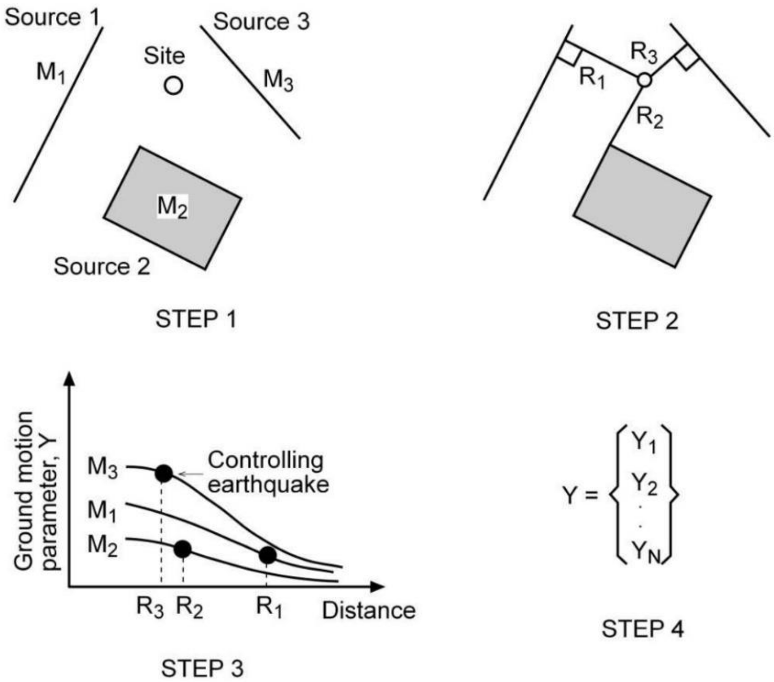


Fig. II.1: Diagrams showing four steps of deterministic seismic hazard analysis (Gürboğa and Sarp 2013)

II.2.2 Probabilistic seismic hazard analysis (PSHA)

Probabilistic seismic hazard analysis (PSHA) can be viewed as the assessment of an infinite number of deterministic hazard analyses taking into account all possible earthquakes which have occurred or can occur in the specified region for all possible scenarios of magnitude and distance. In the past 20 to 30 years, the use of probabilistic concepts has allowed uncertainties in the size, location and rate of recurrence of earthquakes and in the variation of ground motion characteristics with earthquake size and location to be explicitly considered in the evaluation of seismic hazards (Gürboğa and Sarp 2013). The PSHA aims to deal with such uncertainties which can be identified, quantified and combined in a rational manner to produce an explicit description distribution of future shaking provide a more complete picture of the seismic hazard.

A probabilistic seismic hazard assessment combines various elements: i) seismic source zones which represent areas of similar seismicity, ii) earthquake recurrence which defines the probability of exceeding a given magnitude for that source zone iii) and the ground motion prediction equations applied for each source zone wherein the total hazard is calculated by the integration of the contributions of each of these zones. The result is a plot of the annual probability of exceedance of accelerations (Gürboğa and Sarp 2013). For summarizing the PSHA, four steps procedure can be described (Fig. II.2, (Gürboğa and Sarp 2013):

1. Identically to the first step of DSHA, all earthquakes sources should be firstly identifying to define the spatial variation of earthquake activity, except that the distribution probability of potential rupture locations within the source must also be characterized (Fig. II.2). These source zones are based on the distribution of observed seismic activity together with geological and tectonic factors and represent areas where the seismicity is assumed to be homogenous; i.e. there is an equal chance that a given earthquake will occur at any point in the zone.

2. Next, the seismicity or temporal distribution of earthquake recurrence must be characterized to define the level of activity within a particular source zone. The recurrence relationship may accommodate the maximum size earthquake, but it does not limit consideration to that earthquake, as DSHA often do (Fig. II.1). There are, generally more small (low-magnitude) earthquakes than large (higher magnitude) earthquakes. Again observed seismicity is used to determine the earthquake recurrence relationships.

3. Ground-motion predictive equations (GMPEs) to define ground motion produced by earthquakes of any possible size occurring at any possible point in each source zone (define what ground motion should be expected at location A due to an earthquake of known magnitude at location B). Generally, ground-motion predictive equations are derived from past earthquake observations and also provide a measure of the variability of the ground motion parameter. The uncertainty inherent in the predictive relationship is also considered in a PSHA.

4. Finally, the uncertainties in earthquake location, earthquake size, and ground motion parameter prediction are combined using the calculation known as the total probability theorem to obtain the probability that the ground motion parameter can be exceeded during a particular time period.

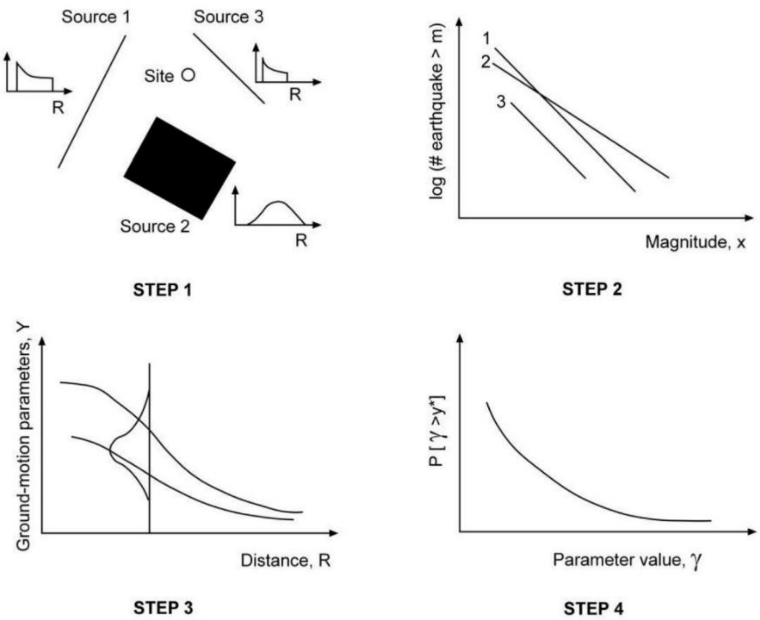


Fig. II.2: Diagrams showing four steps of probabilistic seismic hazard analysis (Gürboğa and Sarp 2013)

II.3 Deterministic versus probabilistic approaches

Both probabilistic and deterministic methods have a role in seismic hazard and risk analyses. These two methods have differences, advantages, and disadvantages that often make the use of one advantageous over the other. Although different in methodology, these two methods can complement one another to provide additional insights to the seismic hazard or risk problem. In this context, deterministic events can be checked with a probabilistic analysis to ensure that the event is realistic (focus on a single earthquake), and probabilistic analyses can be checked with deterministic events to see that rational, realistic hypotheses of concern have been included in the analyses (McGuire 2001).

In any relevant seismic hazard or risk analysis the result will be used to make a decision. One method will have priority over the other, depending on how quantitative are the decisions to be made, depending on the seismic environment (whether the location is in a high, moderate, or low seismic risk region), and depending on the scope of the project (whether one is assessing a site risk, a multi-site risk, or risk to a region) (McGuire 2001). This might be the selection of design or retrofit criteria and levels, financial planning for earthquake losses (levels of insurance or reinsurance, or self-insurance), investments for redundant industrial systems, planning for emergency response and post-earthquake recovery, and planning for long-term recovery (McGuire 2001). Details of the factors of choice and how they are considered by deterministic and probabilistic methods are presented in the Table II.1, (McGuire 2001)

Table II.1: Representative applications of DSHA and PSHA approaches (McGuire 2001)

Decision	Quantitative aspects of decision	Predominant approach
Seismic design levels	Highly quantitative	Probabilistic
Retrofit design	Highly quantitative	Probabilistic
Insurance/reinsurance	Highly quantitative	Probabilistic
Design of redundant industrial systems	Quantitative or qualitative	Both
Training and plans for emergency response	Mostly qualitative	Deterministic
Plans for post-earthquake recovery	Mostly qualitative	Deterministic
Plans for long-term recovery, local	Mostly qualitative	Deterministic
Plans for long-term recovery, regional	Mostly qualitative	Probabilistic

II.4 Seismic activity of the northern of Algeria

II.4.1 Seismic events

In recent years, the interest of the scientific community regarding seismology and seismo-tectonics has greatly increased in Algeria, especially in the fields related to the seismic risk assessment and its possible reduction. As already stated, a good starting point towards the assessment of such risk is the seismic hazard of a site, which should be begun by the study of historical earthquakes and active sources.

Algeria is one of the most seismically active areas in the Mediterranean basin. Often, the seismicity of Algeria is characterized by shallow earthquakes, it is located in the first 20 kilometers. This seismicity is usually marked by low to moderate earthquakes. However, there were strong earthquakes occurred in the Tell Atlas striking different regions, such as those located in the vicinity of Algiers, which occurred on January 2, 1365 (IX), February 3, 1716 (XI), December 3, 1735 (VII), March 17, 1756 (VIII), November 8, 1802 (VIII), June 18, 1847 (VIII) and November 5, 1924 (VIII), and those located in the vicinity of the city of Oran, on October 9, 1790 (IX–X) and May 21, 1889 (VIII). The Djidjelli earthquake (1856, I_o = IX), Orléansville (1854, M_s 6.7) (Benouar 1994). Moreover, for the last 50 years, the El Asnam region has suffered the most destructive and damaging earthquakes recorded in northern Algeria, namely those of September 9, 1954 (M_s 6.8) and October 10, 1980 (M_s 7.3). Also in Constantine (1985, M_s 5.9; (Bounif *et al.* 1987), Tipasa-Chenoua (1989, M_s 6.0), Mascara (1994, M_s 6.0). Moreover the well-known earthquake of Boumerdès on May 21, 2003, which was a destructive earthquake of M_w = 6.8 (Ayadi and Bezzeghoud 2014).

Fig. II.3 illustrates the updated seismic catalog of Algeria: from 1910 to 2013 carried out by the national center of research in astronomy, Astrophysics and Geophysics (CRAAG).

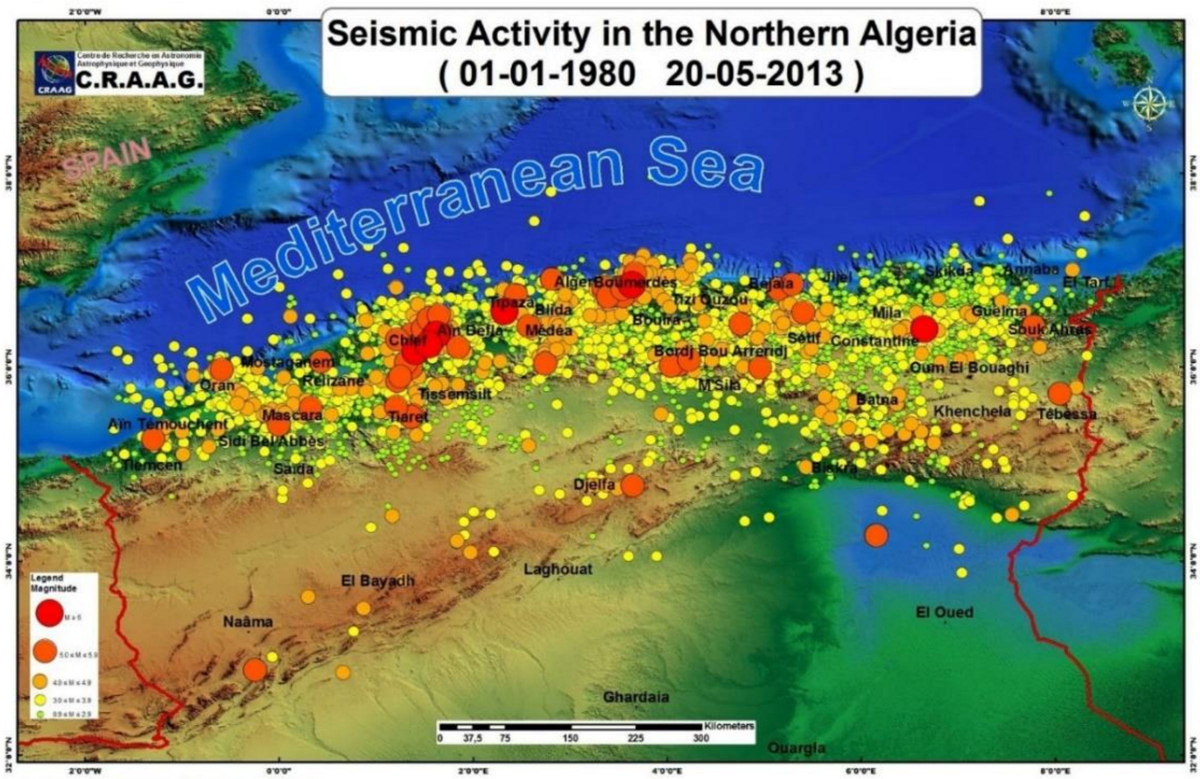


Fig. II.3: The seismic activity in north of Algeria from 1980 to 2013 (CRAAG)

II.4.2 Active faults

The evidence for fault and fold development and on the style of deformation (compressive, strike-slip, …), have been determined on the basis of data collected during the two oceanographic surveys Maradja, 2003; Maradja2/Samra, 2005 which covered from west to east the entire Algerian margin. Different types of high-resolution data have been recorded by instruments on board. Fig. II.4 depicts the different active faults characterized in the north of Algeria (for more details see (Kherroubi *et al.* 2009)).

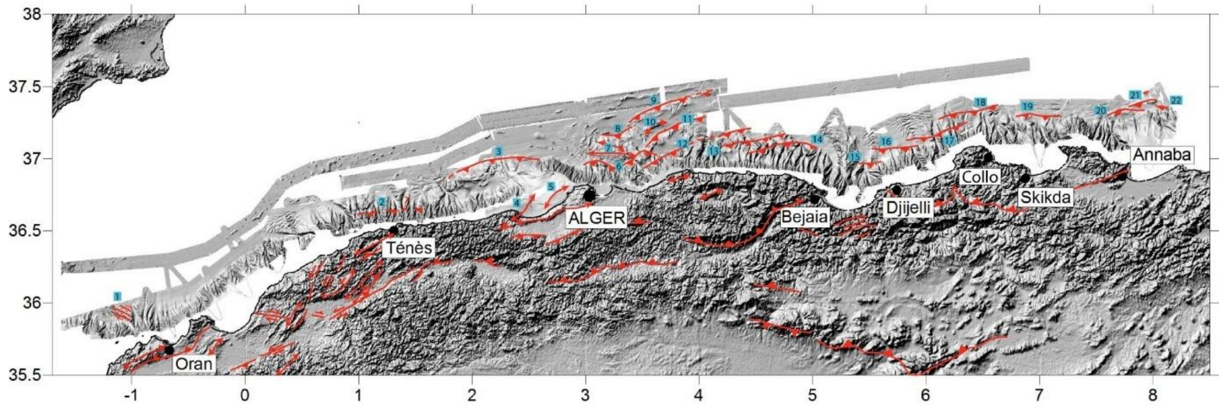


Fig. II.4: The active faults in the north of Algeria (Kherroubi *et al*. 2009)

II.4.3 Macroseismic intensity

Map of historical maximum intensities is a compilation of induced effects on the ground by important historical earthquakes in or near an area. The effects of the largest earthquakes on the ground resulted in maximum intensity observations and represented by isoseismals cards.

The map of maximum intensities (Fig. II.5) carried by Bezzeghoud (1996) was used as the reference card because it shows the different areas of high, medium and low seismic risk all over Algeria especially the northern part. It is achieved on the basis of macroseismic data available for all Algerians earthquakes from 1716 to 1989.

Earthquakes collected for the completion of the map of maximum intensities are intensities exceeding V. Indeed, it is accepted according to the MSK scale and updating EMS and the MM scale, a felt movement is considered strong from an intensity V. The light damages begin to I_0 = VI and significant damage is to I_0 = VIII; an intensity I_0 = IX means that the earthquake is destructive.

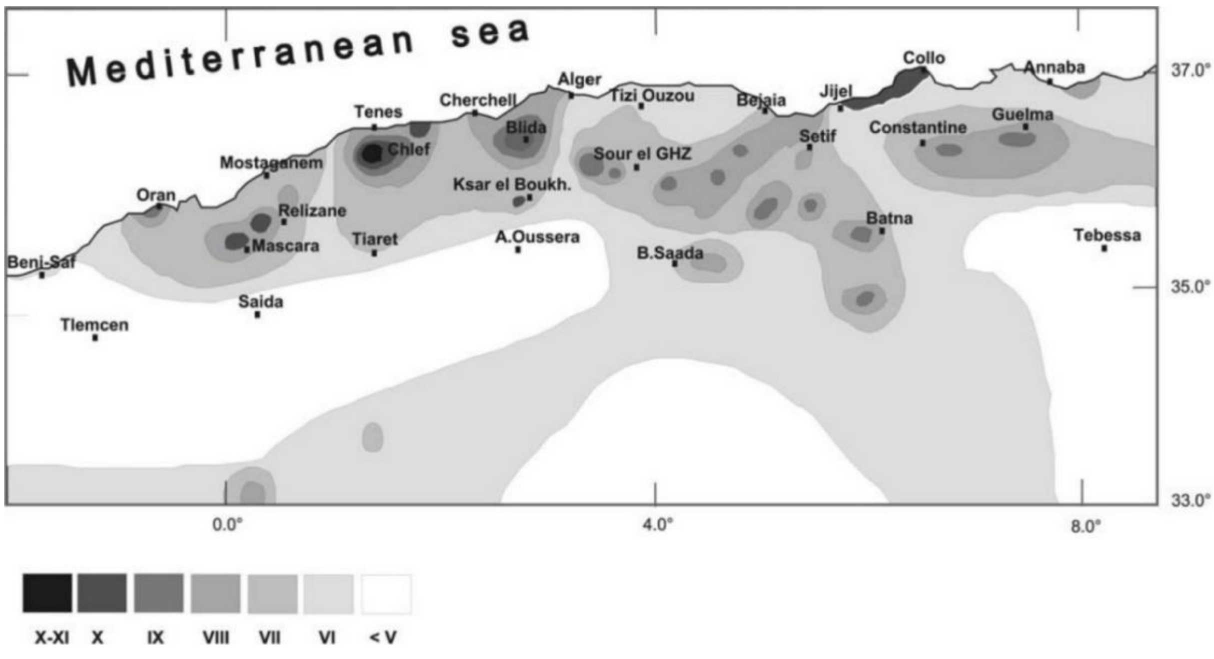


Fig. II.5: Map of maximum intensities for 1716-1989 (Bezzeghoud 1996).

Before 1900, numerous authors conducted seismic studies following the macroseismic approach by evaluating the intensity in relation to the damage produced and the effects generated by the event (Ayadi and Bezzeghoud 2015). Isoseismal curves were drawn for each earthquake showing the extent of damage near the epicenter and the attenuation of the macroseismic intensity. An updated version of the latest one from Bezzeghoud (1996) (Fig. II.5) incorporated all the data available between 1365 and 2013, including also the strong events of the last two decades, such as Mascara (1994), Ain Temouchent (1999), Zemmouri (2003), Laalam (2001), and Beni Ilmane (2010) is performed by Ayadi and Bezzeghoud (2015). More than a thousand intensity data points were used in this study, however it considered only the maximum observed intensity (MOI) for each earthquake, which enabled to draw a map of seismic zonation that highlights the regions of high, medium, and low levels of seismic shaking in Algeria (Ayadi and Bezzeghoud 2015) (Fig. II.6).

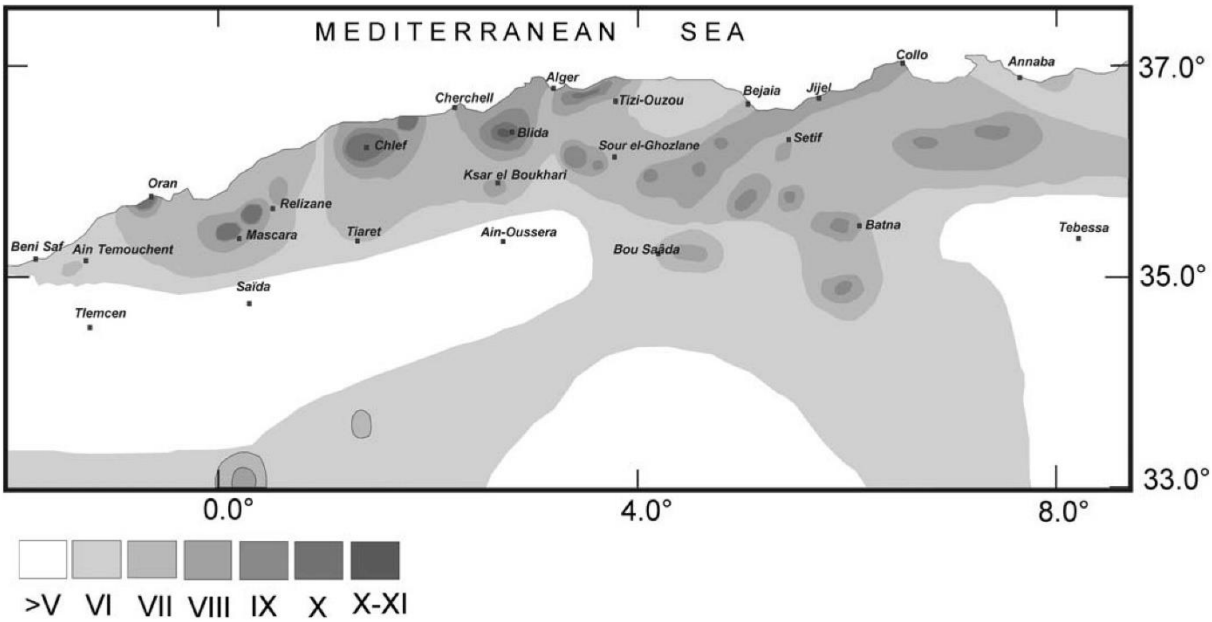


Fig. II.6: Maximum observed intensity (MOI) map of north of Algeria (2014) (Ayadi and Bezzeghoud 2015)

II.5 Seismic activity in the north-eastern of Algeria

II.5.1 Seismic events

The seismicity of the northeast of Algeria is particularly concentrated in the region of Constantine and Guelma (see Fig. II.7). Indeed, for the region of Constantine, at least four earthquakes of maximum intensity VIII were recorded after 1900 (CGS 2011). The first occurred on September 16, 1907; the second on August 4, 1908, with a magnitude M_s = 5.2; and the third on August 6, 1947, with a magnitude M_s = 5.0 (CGS 2011). Furthermore, on October 17, 1985, the city of Constantine was shaken by the fourth and the strongest earthquake of magnitude M_s = 6.0 with maximum intensity (VIII to IX) (Bounif *et al.* 1987). In surrounding villages, the greatest damage has been recorded mainly in houses and farms, which led to the death of five peoples and the injury of more than 300 (Bounif *et al.* 1987).

Regarding Guelma region, the strongest known earthquake is the one that occurred on February 10, 1937, with a maximum intensity of VIII and a magnitude of M_s = 5.2. According to Benouar (1994), it occurred very near to two earlier destructive shocks that occurred in 1908 and 1928, which had caused major damages in the same zone. The earthquake was felt in an extensive area, from the eastern part of Constantine to Tabarca in Tunisia.

At sea, active seismicity is observed in north of Skikda. The most known undersea earthquake is that of Djidjelli of August 22, 1856 of VIII intensity according to the EMS scale (Harbi 2006, Harbi *et al.* 2011). Note also that located in north of Philippeville (Skikda) on September 19, 1935, of intensity VI EMS and magnitude M_s = 4.9 and that of the NE of Annaba on March 20, 1962 of intensity VII EMS and magnitude M_s = 4.9 and finally the NE earthquake of El Kala of intensity VI EMS and magnitude M_s = 5.0 (CGS 2011). Fig. II.7 presents the spatial distribution of the earthquake events in north-eastern Algeria based on the max seismic (Harbi et al. 2010).

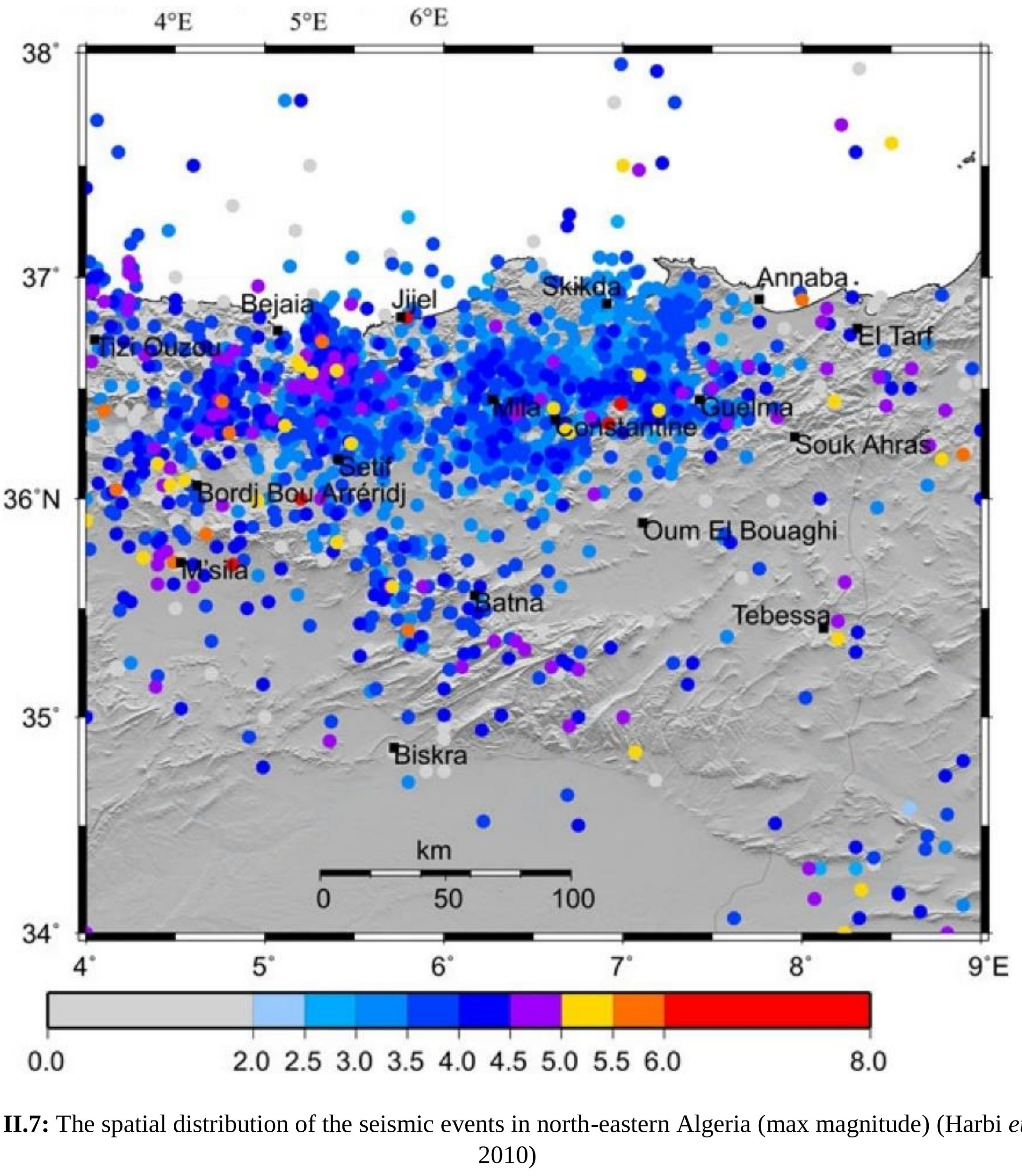


Fig. II.7: The spatial distribution of the seismic events in north-eastern Algeria (max magnitude) (Harbi *et al.* 2010)

II.5.2 Active faults

The data from the seismic history, the photo-geological analysis, field investigations and new data published on Neogene and Quaternary deposits of the northeast part of Algeria have helped to highlight a number of active and probably active faults (Fig. II.4). The geometrical characteristics of these faults are summarized in Table II.2 by the Algerian National Earthquake Engineering Centre (CGS 2011):

Table II.2: Seismic sources lines in the northeastern of Algeria and their characteristics (CGS 2011)

Name of the fault	Type	Length	Direction	Pendage	Depth	M_w
Fault of Ain Smara	Sinistral strike-slip	65km	NE-SW	Vertical	15km	7
Front Constantine tablecloths	Reverse	55km	E-W	50°N	15km	7
Fault of Temlouka	Sinistral strike-slip	25km	NE-SW	Vertical	10km	6
Fault of Sigus	Reverse	60km	E-S	60°N	15km	7
Fault of north of Guelma	Dextral strike-slip	33km	E-S	Vertical	15km	6.7
Fault of south of Guelma	Sinistral strike-slip	30km	NE-SW to E-W	Vertical	15km	6.7
Fault of Hammam N'Bails	Reverse	13km	NE-SW	45°NW	10km	6.5
Fault of Bouchougouf	Strike-slip	18km	NE-SW	Vertical	10km	6.3
Fault of Sebhket Djendli	Reverse	12km	NE-SW	40°N	10km	6.4
Fault of Djebel Youcef	Reverse	26km	NE-SW to ENE-WSW	60-70°S	10km	6.5
Fault of north Djemila	Reverse	45km	NE-SW	50°NW	15km	7
Fault of Jijel Sea	Reverse	100km	NE-SW	40°NW	18km	7.4
Fault of Annaba Sea	Reverse	80km	E-W	40°S	18km	7.3

II.5.3 Macroseismic intensity

As already stated, the macroseismic intensity in the northeast of Algeria varies between low (such as Annaba city), and moderate intensity for the case of Skikda, Guelma and Constantine city. Regarding the two latter cities, the research outcomes proved that the middle of both the cities known by a very important seismicity wherein the macroseismic intensity is assumed to be very significant I ≥ VII according to EMS-98 scale. This fact is due to the high concentration of the active faults which are already listed in the table above. Fig. II.8 illustrates the maximum intensities of the north-eastern of Algeria carried out by CGS (2011).

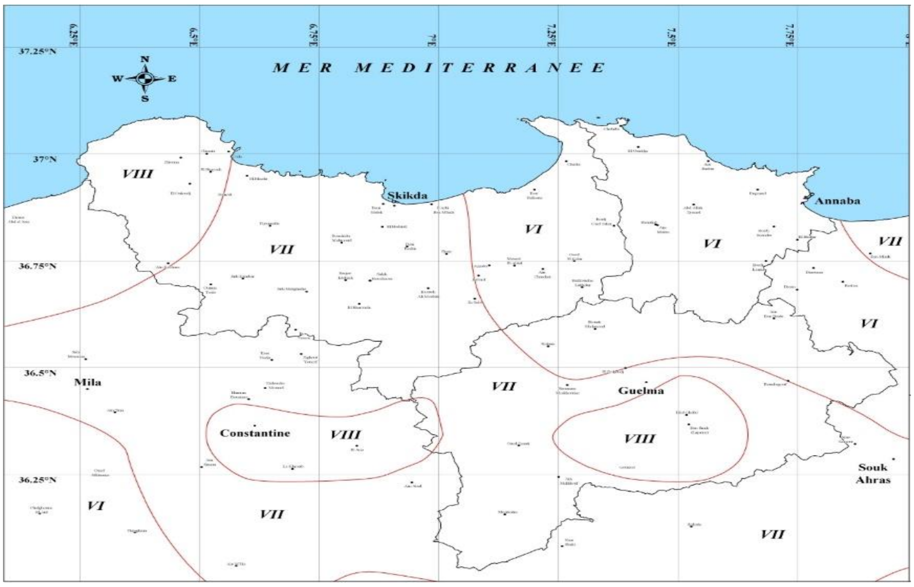


Fig. II.8: Maximum intensities of the north-easthern of Algeria (CGS 2011)

II.6 Seismic activity of Annaba city

II.6.1 Seismic events

Annaba, the fourth important city in Algeria, belongs to a seismogenic zone is known by its low to moderate active seismicity (Mourabit *et al.* 2014). The only seismic event strongly felt in the region, on a quite large area of perceptibility, is the offshore earthquake of Herbillon, which occurred on September 19, 1935 (Harbi and Maouche 2009). Little is known about the tectonic activity of the region with regard to the activity of Eastern Algeria (Kherroubi *et al.* 2009). Annaba is close to two active seismogenic zones Guelma and Constantine (Harbi *et al.* 2003, Mourabit *et al.* 2014). As already mentioned, both zones experienced damaging earthquakes (Benouar 1994, Harbi *et al.* 2010). Furthermore, several seismic events that occurred in the surrounding region may have strong to damaging effects on Annaba city, as can be concluded from the analysis of Table II.3, where the epicentral intensity of the most important historical earthquakes felt in this area and their effect registered in Annaba city in term of intensity according to the European Macroseismic Scale (Grünthal 1998) is presented. It is worthwhile noting that two of the most destructive events of Eastern Algeria that occurred at Djidjelli (Jijel now) on 21 and 22 August 1856 and triggered tsunamis had effects on Annaba city. The shock of 21 August caused some cracks to the theatre and many private dwellings at Bône (Annaba now); the sea had withdrawn a meter, and it was reported that the "Islands of three brothers" located offshore Bône had almost disappeared under the waves (see (Harbi *et al.* 2011) for more details). During the 22 August event the damage in Annaba was limited to the collapse of a hotel, some chimneys and cracks in some dwellings and the sea rose by one meter and flooded during twelve hours a part of the "Champs de manœuvre" (Harbi *et al.* 2011). All these factors have to be taken into account for a better seismic hazard assessment and a reliable mitigation of seismic risk in the region.

Table II.3: List of the earthquakes that have strong to damaging effects at Annaba

Earthquake	Epicentral Intensity (I_0, EMS)	Intensity at Annaba (I, EMS)	Reference
Djidjelli, 21/8/1856	VIII	VI	(Harbi *et al.* 2011)
Djidjelli, 22/8/1856	IX	VI-VII	(Harbi *et al.* 2011)
Constantine, 4/8/1908	VIII	V	Benouar (1994)
Guelma, 10/2/1937	VIII	V	Benouar (1994)
Jemmapes, 5/3/1960	V	IV-V	Harbi and Maouche (2009)

II.6.2 Active faults

The detailed morphology of the margin in the region of Skikda and Annaba is known from the seismic and bathymetric data of Maradja2 companion (2005) whose coverage was limited to the continental slope and the deep basin (Kherroubi *et al.* 2009). In term of active faults (Fig. 1), three segments of a thrust fault with similar length prevail, S1 (30 km length), S2 (30 km length) and S3 (20 km length) (CGS 2011, Kherroubi *et al.* 2009). Based on the scaling relations between the surface displacements and the length (Bonilla et al. 1984; Wells and Coppersmith 1994; Leonard 2010), if these submarine fault segments rupture during a single event, a magnitude of M_w = 7.5 is expected. This value may be even higher if the segment S4 is also considered (Kherroubi *et al.* 2009) (Fig. II.9).

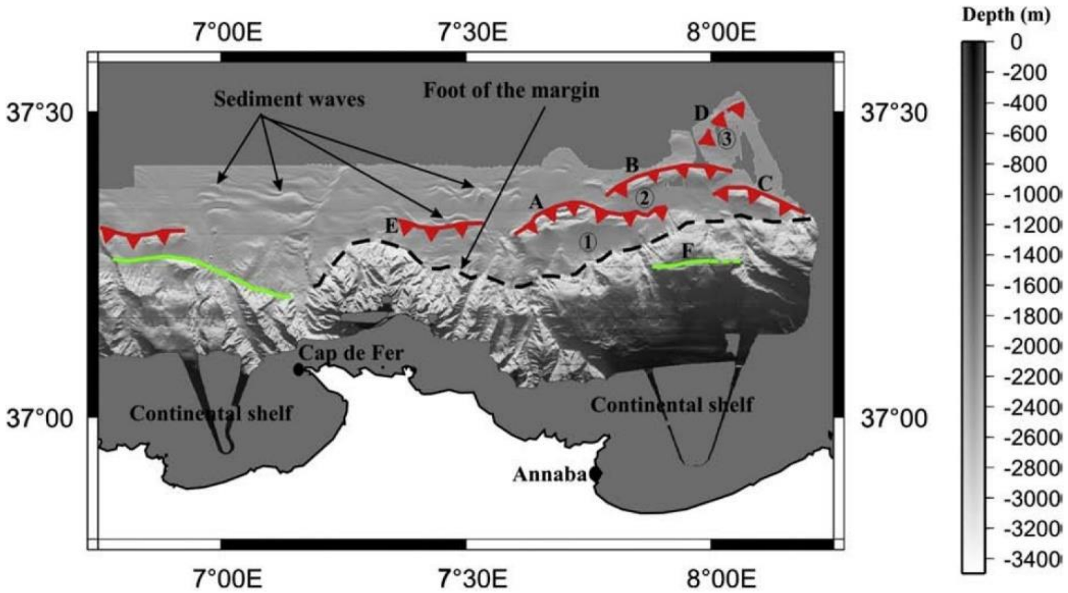


Fig. II.9: Active faults of Annaba Sea (Kherroubi *et al*. 2009)

II.6.3 Macroseismic intensity

It is worth mentioning that the largest event reported of intensity X, occurred in 1722 north of Seraidi (Annaba city) (Harbi 2006) remains uncertain in fact that damages reported by historical documents are limited. Furthermore, most events located offshore suffer possible large uncertainties (Kherroubi *et al.* 2009). In this context, in addition to the lack of information about the consequences experienced on the buildings stock of Annaba city, all these factors lead to an underestimation of the peak intensity on the whole area (CGS 2011), and therefore, a moderate to low seismic hazard is assigned. Indeed, the deductions of Table II.3 is supported by Yelles-Chaouche *et al.* (2006), who assumed that the intensity allocated generally is VI (Fig. II.8) and does not exceed VII.

II.7 Seismic hazard assessment of Annaba city

II.7.1 Probabilistic estimation

According to the framework of Probabilistic Seismic Hazard Assessment (PSHA) of Annaba region performed by the National Earthquake Engineering Centre (CGS 2011), the mapped amount is the average value of the amplitude of ground motion (Peak Ground Acceleration) associated with three return periods 100, 200, and 500 years (CGS 2011) (Fig. II.10). In addition to the historical seismicity data of Annaba region, this study took also into account the seismological, the geophysical and the geological context. As already mentioned, this mode of representation of the seismic hazard by the probabilistic approach is widely responding across the world. The results of this seismic hazard estimation (Fig. II.10) show that the maximum expected acceleration values for the whole study area of Annaba city varies between 0.14g and 0.28g for return periods of 100 and 500 years respectively (CGS 2011).

The plotted map (Fig. II.10) represent the iso-acceleration or acceleration curves (CGS 2011) that count the likely level of activity in Annaba region which is usually the basic tool for seismic zoning, seismic regulations, seismic microzonation and an essential input for the engineer in the seismic design of structures.

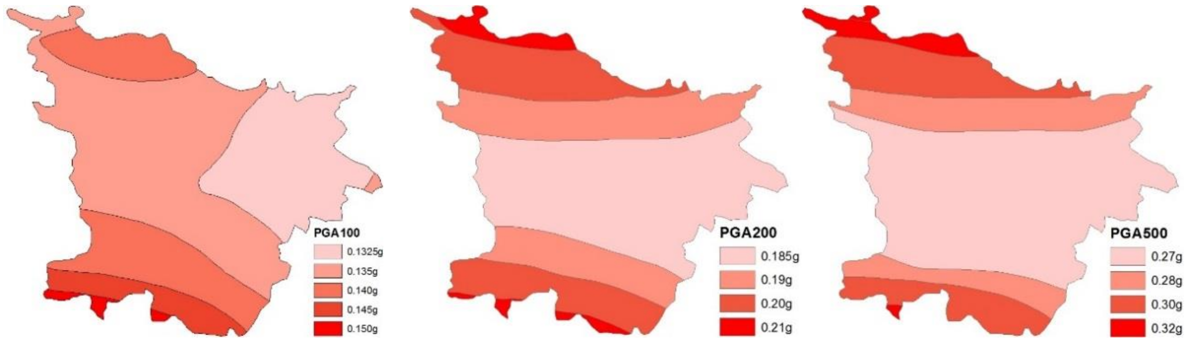


Fig. II.10: Seismic hazard maps of Annaba city in terms of PGA for 100, 200 and 500 return periods (CGS 2011).

II.7.2 Deterministic estimation

For the Deterministic Seismic Hazard Assessment (DSHA) of Annaba city, the ELER (Earthquake Loss Estimation Routine) v.3.0 software intended for rapid estimation of earthquake shaking and losses in the Euro-Mediterranean region, which has been developed in the framework of EU FP-6 NERIES (Network of Research Infrastructures for European Seismology) Project (Hancilar *et al.* 2010) is used. Without going to details, as in the USGS ShakeMap, the ELER Earthquake Hazard Assessment (EHA) module uses earthquake epicenter, magnitude and if available fault information as input. To generate ground shaking intensity maps and maps with parameters of PGA, PGV, S_a and S_d at 0.2, 1.0 and 3.0s, the

Seismic hazard is deterministically computed based on empirical ground motion and/or macroseismic intensity prediction models.

Fig. II.11 presents the DSHA for Annaba city considering the active sea faults (Fig. II.9) and the significant seismic event located close to the old town (area under study). The results are shown in terms of peak ground acceleration (PGA).

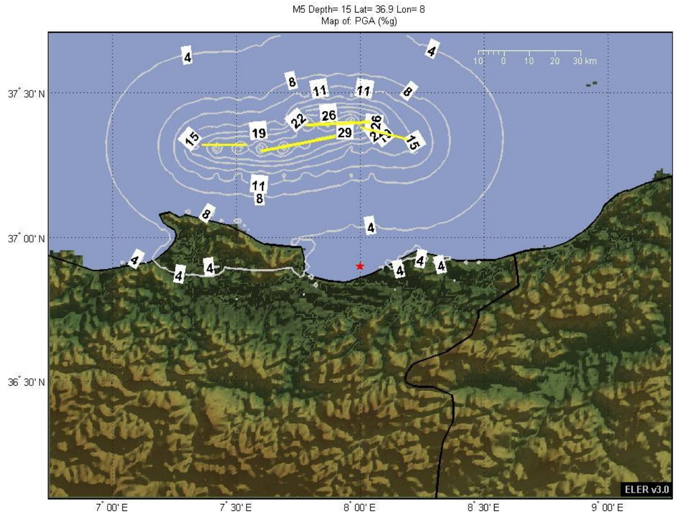


Fig. II.11: DSHA for Annaba city

In the next step, and based on the same previous estimation of the seismic hazard (Fig. II.11), additionally we took into consideration the active faults that located in Guelma city. In the same way, Fig. II.12 illustrates the results in terms of peak ground acceleration (PGA) for the hole deemed region.

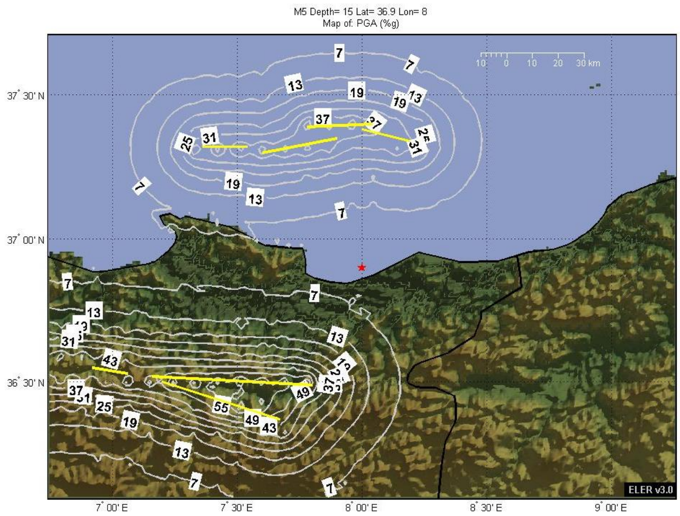


Fig. II.12: DSHA for Annaba region

Compared by the PSHA of CGS, it is clearly noted that more accounting huge number of seismic events and the possible existence of different active faults surrounding the target area, accurate and reliable outcomes can be obtained. Therefore, this fact proves the reliability of the probabilistic seismic hazard assessment principle (PSHA).

Chapter III: Seismic vulnerability

III.1 Introduction

In engineering, vulnerability analysis can be carried out mainly for buildings, essential facilities and lifelines. In this thesis, we focused on the most important issue at present time and the widely used concept in vulnerability assessment works that is related to the protection of buildings against seismic events. Nevertheless, there is not a rigorous accepted definition of it. In general terms, vulnerability measures the amount of damage caused by an earthquake of given intensity over a structure. However, “amount of damage” and “seismic intensity” are concepts without a clear and rigorous numerical definition (Preciado *et al.* 2008) in fact that only over the last century when vulnerability studies are carried out for the purpose of assessing the need to strength the exposed elements against future earthquakes. It is worthwhile noting that nowadays, vulnerability assessment of existing structures is an issue that has been raised and quickly progressed.

Depending on the buildings, whose vulnerability is going to be assessed, different approaches can be used. The ideal method is to apply a series of statistical analysis studies on a sufficient number of samples of similar issues which are subjected to the same seismic performance. Unfortunately, such a case is rare, and often there are no damage databases or they are incomplete. To achieve this goal, all different parts of available data, which are related to the studied subjects should be exploited. Imperfect information of damages should complete by experimental data, the numerical prediction or seismic behavior analysis, and obtained information during field studies. Incorporating all this information, seismic vulnerability prediction based on statistical method will result in satisfactory results (Mehran and Davood 2014).

III.2 Seismic vulnerability assessment methodologies for buildings

Several methods for the vulnerability assessment have been developed and proposed in recent years, however, the selection of a suitable methodology for the development of the seismic vulnerability evaluation of buildings mainly depends on the next aspects: nature and objective of the study, available information, characteristics of the building or group of buildings under study, suitable methodology of assessment (qualitative or quantitative) and the organism which will receive the results of the study (e.g. government, scientific organizations, companies and so on) (Adolfo 2011).

It has been assumed when assessing the seismic vulnerability of a large group of buildings roughly in a quite general manner, simple methodologies should be followed, or to only evaluate one building in a detailed way by means of refined methodologies (Adolfo 2011). Therefore, the analyzed classification criteria agree on the accordance between the chosen vulnerability method and the space scale considered for analysis (e.g. urban level or building level, etc.). In the following, the different approaches are outlined in order of increasing computational effort starting from observed vulnerability and expert opinions, via simple analytical models and score assignments to detailed analysis procedures (Fig. III.1).

Expenditure	Increasing computational effort →				
Application	Building stock				Individual building
Method	Observed Vulnerability	Expert opinion	Simple analytical models	Score assignments	Detailed analysis procedure

Fig. III.1: The different seismic vulnerability assessment methods and their applicability scales (Adolfo 2011)

The oldest method of seismic vulnerability assessment is denoted as empirical approach (also called observed vulnerability; bold line in Fig. III.2) is mainly used when a group of buildings (large scale) are studied and it is based usually on empirical methodologies consisting in assessing vulnerability from observations of statistical damage distributions due to past earthquakes (Fig. III.2) (Calvi and Pinho 2006).

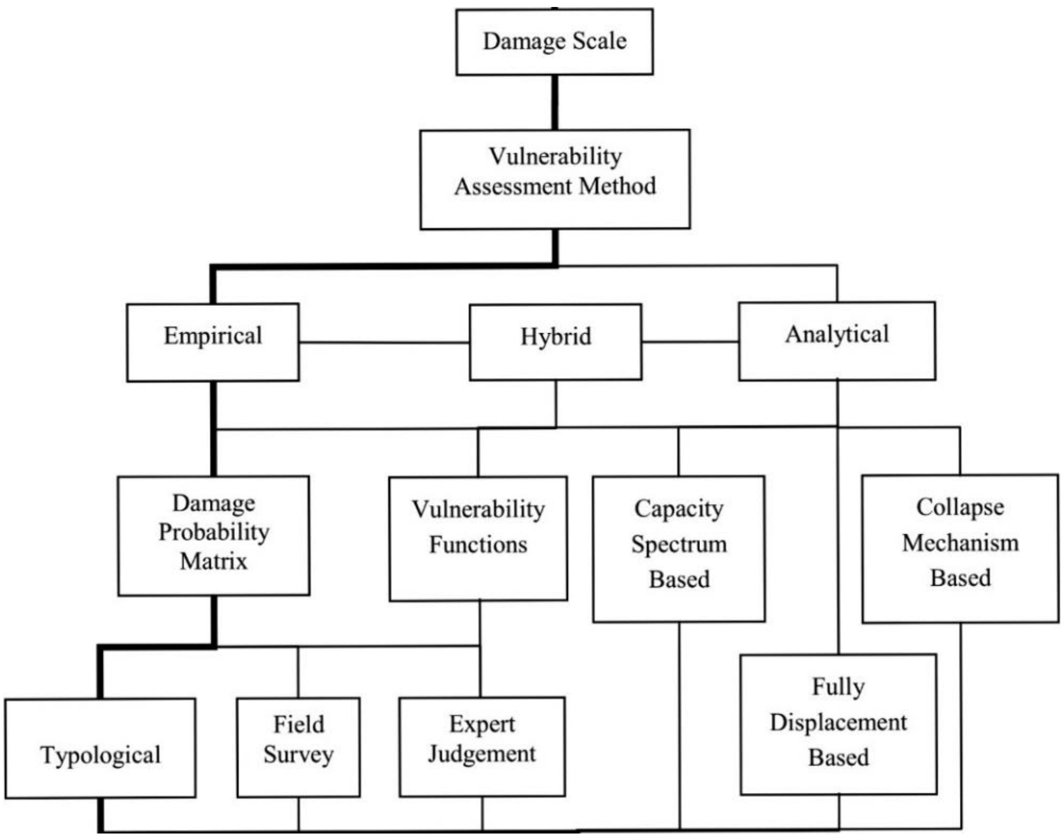


Fig. III.2 : The choices for the seismic vulnerability assessment procedure; the bold path shows a traditional assessment method (Calvi and Pinho 2006)

The overall seismic vulnerability methodologies presented in Fig. III.2 allow obtaining a qualification of the assessed buildings either in numerical or in qualitative terms that could range from low to high. However, often the data of the first category of approaches (observational methods) are limited and do not concern all the building typologies and all the intensities that it would be necessary to represent in a vulnerability model. For this reason, the processing of the observed data at the root of observational methods is often supported by expert judgment or analytical approaches (ENSURE 2009). This latter is considered herein as a second category of vulnerability assessment methods (Fig. III.2). The analytical methods is especially used to deal a single structural units (local scale) and it refers to the assessment of the expected building performance based on calculation and design specifications by considering its individual features, as well as local soil characteristics, and using some detailed numerical analyses. Consequently, the mixture between the two stated approaches leading to a third one denoted as hybrid approach (Fig. III.2).

III.2.1 Empirical vulnerability methods

The seismic vulnerability assessment of buildings at large geographical scales has been first carried out in the early 70's, through the employment of empirical methods initially developed and calibrated as a function of macroseismic intensities (Calvi *et al.* 2009). As a matter of fact that in earlier times intensities were the only measure of earthquake shaking, recording stations were not yet available and thus instrumental earthquake records were less common. Even today, the lack of recording stations or their widespread placing in many earthquake-prone regions prohibits the conduct of earthquake loss studies based on physical parameters (Lang 2012). Therefore, the empirical methods are the only reasonable and possible approach that could be initially employed to perform a preliminary evaluation of a building or a large group of buildings at territorial scale in a fast way when the available information is limited.

As already indicated, empirical approaches refers to earthquake loss studies based on datasets of observed damage. In general, post-earthquake explorations at the corresponding site are the main source of these datasets where the effects to structures correlating recorded damage with an estimated ground motion level. These qualitative evaluations are commonly developed in-situ by means of a questionnaire of evaluation and visual inspections especially suitable for non-engineered structures made of low-strength materials such as timber and unreinforced masonry whose earthquake resistance is rather difficult to calculate (Lang 2012).

Within this first category, the vulnerability of the buildings is usually represented either in terms of Damage Probability Matrices (DPM) or Vulnerability (Fragility) curves. The latter represents the relationship between the probability of damage occurrence and increasing ground motion severity in a continuous way, however the DPM describe it in a discrete manner. In certain cases, the lack of high-quality observational datasets means that some of the most commonly used sets of fragility curves partly (if not, extensively) should supplemented with expert opinion (Porter and Scawthorn, 2007).

III.2.2 Damage Probability Matrices

This method of assessment is based on the statistics of the buildings' damage from the past earthquakes. The method is specifically suitable for poor quality non-engineered construction whose resistance is difficult to calculate by analytical or numerical methods. One of the first to have systematically compiled statistics on damage to buildings from experiences during earthquakes was Whitman *et al.* (1973) from a survey of damage caused by the San Fernando earthquake of 9 February 1971 covering approximately 1600 buildings (Lang 2002). The statistical results were presented in the form of a damage probability matrix (DPM). Although the observational source is the most realistic, the data are problematic due to inaccuracies, incoherence and subjectivity associated with building types, damage states, ground motion descriptions. Therefore the analysis of such data for the area under consideration cannot be extended to other towns and cities. Moreover, the method does not possess the ability to calculate reduction in vulnerability of buildings as a result of their retrofitting or strengthening.

The general form of such a damage probability matrix is shown in Table III.1. Each number in the matrix expresses the probability that a building of a certain building class will experience a particular level of damage as a result of a particular earthquake intensity. The damage ratio is defined as the repair cost as a ratio of the replacement cost at the time of the earthquake (Lang and Bachmann 2003).

Table III.1: Format of the Damage Probability Matrix Proposed by Whitman *et al.* (1973)

Damage State	Structural Damage	Non-structural Damage	Damage Ratio (%)	Intensity of Earthquake				
				V	VI	VII	VIII	IX
0	None	None	0-0.05	10.4	-	-	-	-
1	None	Minor	0.05-0.3	16.4	0.5	-	-	-
2	None	Localized	0.3-1.25	40.0	22.5	-	-	-
3	Not noticeable	Widespread	1.25-3.5	20.0	30.0	2.7	-	-
4	Minor	Substantial	3.5-4.5	13.2	47.1	92.3	58.8	14.7
5	Substantial	Extensive	7.5-20	-	0.2	5.0	41.2	83.0
6	Major	Nearly total	20-65	-	-	-	-	2.3
7	Building condemned		100	-	-	-	-	-
8	Collapse		100	-	-	-	-	-

This format of DPM has become the most widely used form to define the probable distribution of damage, which was also adapted by several other methods, however with less number of damage states, ranging between four and six since too many damage states are rather difficult to distinguish. Indeed, based on the damage data of Italian buildings after the 1980 Irpinia earthquake, the buildings were separated into three vulnerability classes (A, B and C) and a DPM based on the MSK scale was evaluated for each class (Colombi *et al.* 2008). This DPM known by GNDT I level approach which differs from the DPM proposed by Whitman *et al.* (1973). The GNDT damage probability matrix make reference to MCS intensity rather than MMI scale because the Italian seismic catalogue is mainly based on this intensity and describes the damage by means of a five damage grade scale.

Dolce et al. (2003) have also adapted the original matrices wherein an additional vulnerability class D has been included using the EMS98 scale (Grünthal 1998) to allow considering the buildings that have been constructed since 1980. These buildings should be either retrofitted or designed to comply with recent seismic codes in order to decrease their vulnerability.

A macroseismic method has recently been proposed (Giovinazzi and Lagomarsino 2004) that leads to the definition of damage probability functions based on the EMS-98 macroseismic scale (Grünthal 1998). The EMS-98 scale defines qualitative descriptions of “Few”, “Many” and “Most” for five damage grades for the levels of intensity ranging from V to XII for six different classes of decreasing vulnerability (from A to F). Damage matrices involve a qualitative description of the buildings rate that belong to each damage grade for various levels of intensity as presented in Table III.2 (Giovinazzi and Lagomarsino 2004).

Table III.2: Damage matrices for vulnerability classes A-F

	Vulnerability class A				
I	D1	D2	D3	D4	D5
V	Few				
VI	Many	Few			
VII			Many	Few	
VIII				Many	Few
IX					Many
X					Most
XI					
XII					

	Vulnerability class B				
I	D1	D2	D3	D4	D5
V	Few				
VI	Many	Few			
VII		Many	Few		
VIII				Many	Few
IX					Many
X					Most
XI					
XII					

	Vulnerability class C				
I	D1	D2	D3	D4	D5
V					
VI	Few				
VII		Few			
VIII		Many	Few		
IX			Many	Few	
X				Many	Few
XI					Many
XII					Most

	Vulnerability class D				
I	D1	D2	D3	D4	D5
V					
VI					
VII	Few				
VIII		Few			
IX		Many	Few		
X			Many	Few	
XI				Many	Few
XII					Most

	Vulnerability class E				
I	D1	D2	D3	D4	D5
V					
VI					
VII					
VIII					
IX		Few			
X		Many	Few		
XI			Many	Few	
XII					

	Vulnerability class F				
I	D1	D2	D3	D4	D5
V					
VI					
VII					
VIII					
IX					
X		Few			
XI		Many	Few		
XII					

Finally, is worth mentioning that, the Damage Probability Matrices (DPM) can be represented graphically by continuous vulnerability (or fragility) functions (generally curves). As already cited, these curves relate the probability of reaching or exceeding a specific damage state to a given ground motion level, which is generally expressed in terms of macroseismic intensity (e.g. EMS98) or Peak Ground Acceleration (PGA). However, it should be also indicating that such a methodology, which is based on a single-parameter representation for the seismic severity may lead to strong uncertainties in the estimation of damages when compared to observations due to a poor definition of the actual seismic aggression.

III.2.3 Vulnerability Index Methods

The vulnerability index methods are commonly used to identify and to characterize the potential seismic deficiencies of a building or group of buildings. Their qualification usually done by means of points assigned to every significant component having influence in their seismic response such as quality of materials, type of foundations, number of stories, state of conservation, or stiffness of the structure. This allows to user the determination in most cases the so-called seismic vulnerability index. This index is a numerical value attributing to each building representing its "seismic quality".

Depending to the score attributed to each building, in particular, the current methods must quantify the level of damage likely to be sustained according to the severity of ground motion. The main objective of these procedures is to determine whether a particular building should or should not be subjected to a more detailed investigation using the next screening level or some numerical analyses (mechanical approaches). It has an important role to play for prioritizing buildings for seismic retrofit.

The vulnerability index methods are qualified as indirect methods since there is no direct relationship provided between the seismic action and the observed damage; the damage index or score is determined only on the basis of building observations (data collected from surveys, expert judgment). The main advantage of ''indirect'' vulnerability index methods is that they allow the vulnerability characteristics of the building stock under consideration to be determined rather than base on the vulnerability definition of the typology alone. Nevertheless, the methodology still requires expert judgment to be applied in the assessing of buildings because the coefficients and weights contribute in the calculation of the final vulnerability index have a degree of uncertainty that is not generally accounted for.

Furthermore, for the vulnerability assessment of buildings on a large scale to be performed using vulnerability indices, a large number of buildings need to be assessed and combined with census data. In the case where such data are not already available, the calculation of the vulnerability index for a large building stock would be very time consuming. However, the approaches based on the principal of vulnerability index are easily implemented within a GIS-based multi-risk analysis, which is generally used to draw up seismic scenarios for urban areas and consist in simulating a single earthquake.

Within the framework of the seismic plan, the CETE and BRGM (Ghislaine 2008) have grouped thirteen methods for assessing the vulnerability of buildings to earthquakes as a guide in which they are compared with the evaluation criteria in several areas: the general principles (scientific validation, scope, types of buildings concerned vulnerability factors taken into account), the level of complexity (data needed, technical skills, simplicity) as well as the necessary means for their implementation (time, cost) and the type of results.

One of the most famous methods usually found in the relevant literature corresponds to the methodology developed by Benedetti *et al.* (1988) and the GNDT (1993a). The GNDT (Gruppo Nazionale per la Difesa dai Terremoti) is the vulnerability index "mother method" which has been extensively used in Italy in the past few decades. The corresponding vulnerability model is calibrated on the data from continuous experimentation and observed damage of certain types of structures (mainly unreinforced masonry buildings) after earthquakes of different intensities to get a good correlation between vulnerability index, damage and macroseismic intensity or PGA (Adolfo 2011).

III.2.4 Analytical approaches

The analytical approach for the seismic vulnerability assessment of buildings may also be called a purely theoretical approach since in contrast to the empirical approach, it is not based on observation, but rather on the numerical methods based on the classical theories of elasticity and plasticity, and in more recent years in the theories of cracking and damage.

These quantitative methods are the most commonly used to evaluate the seismic vulnerability of essential buildings that require special attention such as the case of the seismic protection of historical buildings, hospitals, museums, schools and so on. In fact, many parameters for modeling the real physical characteristics of the actual structures are required, representing with more time consuming.

Generally, within these approaches the building's behavior is expressed in terms of a capacity curve defined on the basis of relationship between the base shear force and the lateral displacement of a control node of the building (Fig. III.3) (Lang 2012). To identify such nonlinear behavior of the structure, a nonlinear structural analysis method such as the famous pseudo-static "pushover" analysis method is required (Lang 2012).

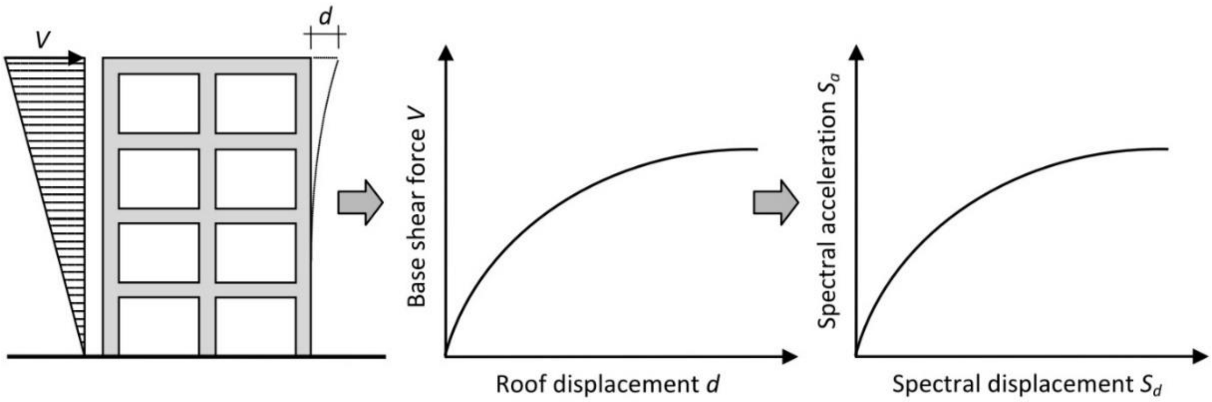


Fig. III.3: Analytical way to generate building capacity curves which ideally represent the nonlinear (damaging) behavior under a statically increasing lateral load (Lang 2012)

The analytical methods do not only allow detailed sensitivity studies to be undertaken, but also cater to straightforward calibration to various characteristics of building stock and hazard. This matter is a definite disadvantage of empirical methods where ground motion can only be represented by a single parameter, e.g. a shaking intensity or PGA. In this fact, the second component of seismic input (or seismic demand) is generally represented by a response spectrum that allows the consideration of the spectral content of ground motion in terms of physical parameters; i.e. accelerations and displacements. In order to be able to correlate the response spectrum with building capacity, these latter should be presented in a compatible spectral domain, thus, the conventional curve (S_a –T) of the response spectrum must be converted into the domain of the capacity curve; i.e. spectral acceleration–spectral displacement domain (S_a –S_d; Fig. III.4) (Lang 2012).

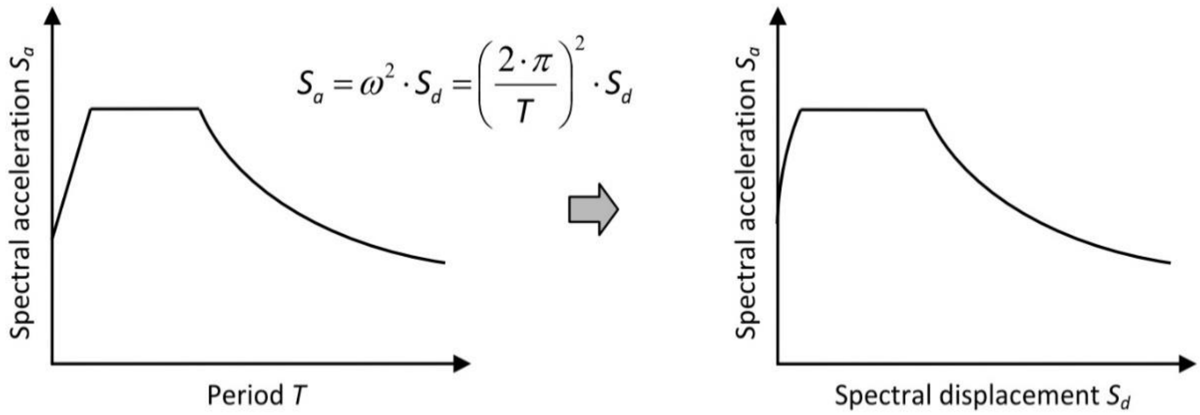


Fig. III.4: Conversion of design response spectrum into S_a–S_d domain (Lang 2012).

Finally, for a building of given capacity subject to a given seismic action, different methods are available to predict analytically its structural damage, noting:

- Capacity Spectrum Methods (CSM)
- Collapse-based methods (CBM)
- Displacement-based methods (DBM)
- Displacement coefficient methods (DCM)
- Incremental dynamic analysis (IDA)

Even though neither of the mentioned procedures will be discussed here in detail, it is worthwhile mentioned that CSM and DCM have received the greatest attention to date, mainly because these procedures were published as various FEMA provisions and, in the case of CSM, because this procedure established the basis for FEMA's HAZUS methodology (Lang 2012).

III.2.5 Hybrid methods

Studies based upon hybrid methodologies are limited and not many studies have been published so far. The seismic vulnerability evaluation of buildings using the hybrid methods consists of a combination of the previous stated methods (empirical, analytical, experimental and/or expert judgment). For instance, after assessing the seismic vulnerability of a group of buildings by the empirical methods, an organized list by level of vulnerability (low, medium and high) could be generated, selecting from it the most vulnerable and important to analyze them by more refined methods such as the analytical-experimental. In this way more reliable results towards the seismic vulnerability of the buildings are obtained.

In the case when observational data are used, the computational effort that required to produce a complete set of analytical vulnerability curves of DPMs would be reduced. This, however, does not necessarily mean that hybrid studies are most relevant for those regions with low earthquake damage experience. A lack of empirical vulnerability studies exists even in many countries with significant seismicity (Lang 2012).

The hybrid methods are usually considered as a convenient tool for the loss estimation studies especially where components of both analytical and empirical methods are used. Even for areas with little empirical data for the geographical area under consideration hybrid models allow calibration of the analytical model to be carried out (Calvi and Pinho 2006).

Kappos *et al.* (1998) emphasize on calibrating analytical fragility curves by available empirical data. Further, based on the work of Whitman *et al.* (1973), they construct parts of the DPMs with respect to intensities, damage grades or building classes for which empirical data are available. Based on the results of nonlinear dynamic analysis of models that simulated the behavior of each building class, the remaining parts of the DPMs were constructed (Dolce *et al.* 2006). Subsequently, in order to correlate the physical results to corresponding intensities empirical correlation relationships are used.

III.3 Literature review on respects to seismic risk analysis requirements

The choice of a suitable method for the vulnerability assessment strictly depends and strongly influences all the components defining the seismic risk analysis (hazard description, exposure characterization and damage evaluation). With regard to the exposure characterization, based on the amount and the quality of the available data of the built system and the importance of the analyzed area, the vulnerability method has to make reference for single building, its typology, its category or its vulnerability class. Considering the hazard analysis, the choice of a vulnerability method usually refers to the seismic input, which could be provided either in terms of a physical parameter or of a macroseismic size depending on the genesis of the selected methodology (observed, expert based or analytical approach). Finally the adverse effect of the seismic event could be expressed by the vulnerability method in terms of the physical state of the built system or directly in terms of losses. Damage Probability Matrices, Vulnerability Curves and Vulnerability Scores allow obtaining a direct evaluation of physical damage while Fragility Curves can possibly provide loss results (Giovinazzi 2005).

Considering the discussion above, it is possible to recognize that the aim characterizes an "optimal" methodology in order to be able to identify the seismic vulnerability of the buildings.

Indeed, the "optimal" approach should:

- take into consideration accurate information in field of seismic hazard assessment
- be able to reduce the uncertainty relative to the chosen approach
- be adaptable for any kind of constructions
- be accurate in the final results.

However, it is difficult to find a methodology covers the entire mentioned features. For this reason in the following sections several adaptations and modifications will be presented upon the selected approaches with the main issue to form a suitable vulnerability assessment approach for our study at urban scale.

III.4 The description of the selected methodologies

III.4.1 GNDT approach

The Gruppo Nazionale per la Difesa dai Terremoti – GNDT is the Italian government research body in charge of the seismic risk evaluation and definition of the required measures to reduce it. In 1984, the GNDT group developed and adapted a method to take account of lessons learned from the subsequent earthquakes of all seismically active regions of Italy with minor modifications. The GNDT method was initially applied in the analysis of several historic city centers in various regions of Italy; for example Catania in 1999 and Molise in 2001 (Vicente *et al.* 2014). The GNDT method involves the determination of a building vulnerability classification system (vulnerability index) via observation of physical construction and structural characteristics based on the identification and in some cases, the calculation of characteristic parameters responsible for the control of the studied building's seismic response. Two complementary approaches have been published by this group, with the aim of being applied in the assessment of the seismic risk in the Italian territory:

- The GNDT I level approach is nothing more than a DPM method, having three classes of vulnerability from A to C, each of these having a DPM. This approach does not lead to the evaluation of a vulnerability index, it requires an external visual inspection of the building based on a unique data record sheet for all structure types. The visit of inside the building is made necessary by the evaluation of eight sections: data on the completed certificate, building location, metrics, condition of finishing and facilities, structural type, extent and level of damage (Ghislaine 2008). These information are quite general and fairly easy to spot on the structure, and the collection of additional data, outfield visit are required to determine the age class of the building, the various interventions that took place on the building (expansion, raising, restructuring ...), the use of the building etc. In some cases, the data of the first level can be used to complete the missing data of the second level. Regarding GNDT I, the seismic demand is considered through the use of the EMS-98 intensity scale and the damage is described by means of a qualitative description according to the level of damage reached by the building. Noting that the description of this methodology can be found elsewhere (GNDT 1993b).

- In the second level, GNDT method distinguishes tow typologies of buildings; masonry and reinforced concrete structures. Likewise level 1, the level 2 involves a visual examination from the exterior and interior. The necessary information is grouped into eleven parameters, nine common to both typologies (typology and organization of the Resistant System, quality of Resistant System, conventional strength, position and building foundations, floors, plan configuration, elevation configuration, non-structural elements and the stat of the building) and two separate (thickness/length and roof for masonry buildings, critical nodes elements and fragile items (regardless ductile) for reinforced concrete buildings) (Ghislaine 2008). The eleven parameters are then combined afterwards to get a vulnerability index I_v as hereinafter described in detail for the masonry buildings.

III.4.1.1 Second level for masonry buildings

The screeners assigns A to D evaluation for each of the eleven parameters, the A mark being more favorable to the good behavior of the structure against seismic loads. The method allows initially to weight the score for each of the eleven criteria to calculate the building's vulnerability index I_v as defined in Eq. (III.1) (Ghislaine 2008). Table III.1 presents the original GNDT level II method with the different weights of the eleven parameters. In some of them, the corresponding weight is put as variable, this means that the parameter varies according to certain specific situations, for more details see (Ghislaine 2008).

$$I_v = \sum_{i=1}^{11} W_{Vi} \times P_i \tag{III.1}$$

Table III.3: Scores and relative weights to compute I_v (Ghislaine 2008)

PARAMETERS		Vulnerability Class				Weight
		A	**B**	**C**	**D**	W_i
P1	Type and layout of resisting system	0	5	20	45	1.00
P2	Quality of resisting system	0	5	25	45	0.25
P3	Conventional strength	0	5	25	45	1.50
P4	Location and soil conditions	0	5	25	45	0.75
P5	Horizontal elements (diaphragms)	0	5	15	45	variable
P6	Configuration in plan	0	5	25	45	0.50
P7	Configuration in height	0	5	25	45	variable
P8	Maximum distance between walls	0	5	25	45	0.25
P9	Roof	0	5	25	45	variable
P10	Non-structural elements	0	0	25	45	0.25
P11	General state of preservation	0	5	25	45	1.00

The vulnerability index is then normalized to obtain a value between 0 and 100.

III.4.1.2 Damage grade estimation

In a second step, damage curves (vulnerability functions) are calculated to enable the formulation (in a fast and simple manner) the damage suffered by buildings for each level of peak ground acceleration (PGA) and vulnerability index. For each value of this latter, the corresponding curve correlating the damage ratio and the demand represented in PGA, by means of a tri-linear curve resembling somehow the so called "fragility curves". The damage is zero until the initial acceleration value y_i and then varies linearly to the acceleration of ruin y_c. For the acceleration values more than y_c, the average damage equal 1. In the case of masonry buildings, y_i and y_c are expressed in the following form (Eqs. III.2 and III.3) (Ghislaine 2008):

$$y_i = \alpha_i \exp\left(-\beta_i V_{building}\right) \tag{III.2}$$

$$y_c = \alpha_c \exp\left(-\beta_c V_{building}^{\gamma}\right) \tag{III.3}$$

With $\alpha_y = 0.18$, $\beta_y = 0.015$, $\alpha_c = 1.0$, $\beta_c = 0.001$ and $\gamma = 1.80$.

Fig. III.5 illustrates the tri-linear form of the vulnerability curves for masonry buildings proposed in the GNDT II level approach that are performed as function of acceleration versus damage ratio (GNDT 1993b).

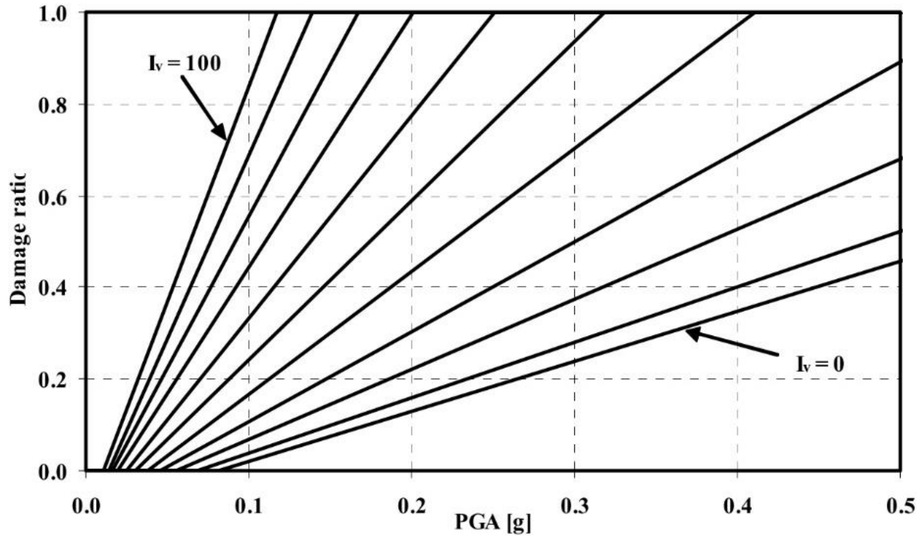


Fig. III.5: Acceleration versus damage ratio tri-linear curves for masonry buildings proposed in the GNDT II level approach (Ghislaine 2008)

Giovanezzi and Lagomarsino (2003) subsequently proposed expression of the average damage not based on peak ground acceleration but the intensity *I* of the earthquake (EMS scale 98). The average damage is then expressed as following (Eq. III.4):

$$D = 0.5 + 0.45 \arctan\left(0.55\left(I_{EMS98} - 10.2 + 0.05 I_v\right)\right) \qquad \text{(III.4)}$$

The calculated damage grade using Eq. (III.4) is directly or indirectly derived from the observed damage wherein the damage scale that they make reference corresponds to the one employed by macroseismic scale (Grünthal 1998). In the modern macroseismic scale, the damage is represented in a discrete form through five damage grades DG_k (k=1÷5). Table III.4 shows the five damage grades of EMS-98 scale (Grünthal 1998) that are used by the macroseimic methods.

Table III.4: Definition of the damage grades according to the macroseismic method

Damage Grade (D_G)	D_{G1}	D_{G2}	D_{G3}	D_{G4}	D_{G5}
Structural Damage	Slight	Moderate	Heavy	Very heavy	Destruction

The assessed damages scaled from 0 to 1, may be transcribed on the EMS-98 scale applying the equivalence presented in Fig. III.6 (Guéguen *et al.* 2007).

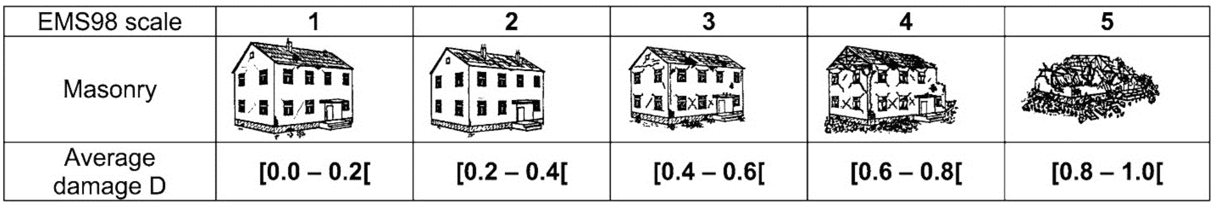


Fig. III.6: Damage equivalence between the EMS-98 scale and GNDT method (Guéguen *et al.* 2007)

III.4.2 RISK-UE LM1 method

This methodology was developed in the framework of a European project by organizations from different European countries (AUTh, BRGM, CIMNE, CLSMEE, IZIIS, UTCB, UNIGE) in the framework of Work Package 4 on the vulnerability assessment of the current buildings. This project aims to analyze the seismic risk at the scale of a city and leads to the creation of a methodology for its assessment. Indeed, the RISK-UE method has been applied in seven cities: Bitola (Macedonia), Thessaloniki (Greece), Catania (Italy), Bucharest (Romania), Barcelona (Spain), Sofia (Bulgaria) and Nice (France).

In the WP4, two methods have been developed to assess the vulnerability of buildings (RISK-UE 2003):

- the first one, in the following referred as a macroseismic method Level 1 or LM1 method, based on the assignment to building a vulnerability index, which is suitable for vulnerability, damage and loss assessments in urban environments having not detailed site seismicity estimates but adequate especially to the seismic intensity.

- the second one, is that referred as a mechanical method, Level 2 or LM2 method, based on analytical analyzes of the structure in dynamic or simplified modeling, usually suitable for urban environments possessing detailed micro seismicity studies expressed in terms of site specific spectral quantities such as spectral acceleration, spectral velocities or spectral displacements.

Due to the difficulties of the mechanical/analytical method stated previously in section III.2.4, in our research work we are not interested by the Level 2 of the RISK-UE method.

III.4.2.1 Masonry building classification

Building Typology Matrix (BTM) systemizing the distinctive features of European current building stock in the countries participating in the RISK-UE method. The classification of the European buildings according to their typology is studied in details in WP1 (Dan *et al.* 2005).

While the RISK-UE BTM (WP1 Handbook) initially consists of 23 building classes (Dan *et al.* 2005) (10 masonry, 7 reinforced concrete, 5 steel and 1 wooden building class), therefore, the BTM prevailing RISK-UE Cities dominant especially by the masonry building types (Table III.5). Regarding this category of buildings, the classification is made by:

- Structural types; and,
- Material of construction.

Three typical height classes make further sub-grouping of such buildings:

- low-rise (1-2 stories);
- mid-rise (3-5 stories);
- high-rise (6+ stories).

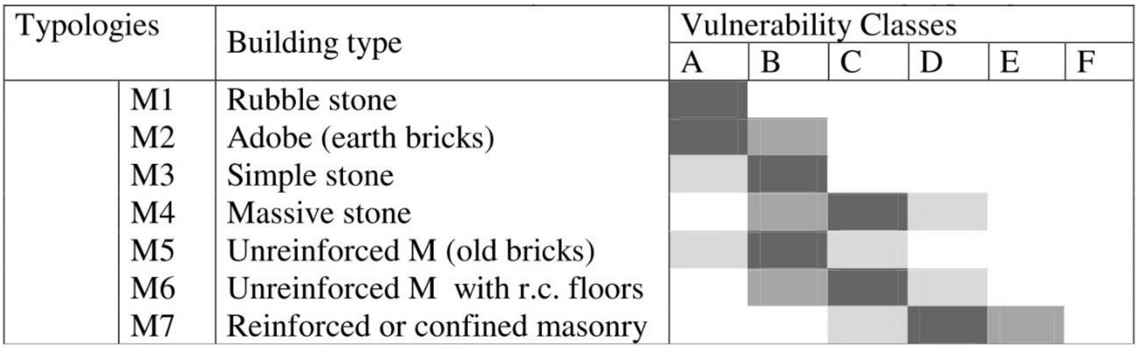
Table III.5: RISK-UE Building Typology Matrix for masonry buildings (Dan *et al*. 2005)

Label	Description	Name	Stories range	Height range
M11L	Rubble stone, fieldstone	Low-Rise	1 - 2	≤ 6
M11M		Mid-Rise	3 - 5	6 - 15
M12L	Simple stone	Low-Rise	1 - 2	≤ 6
M12M		Mid-Rise	3 - 5	6 - 15
M12H		High-Rise	6+	> 15
M13L	Massive stone	Low-Rise	1 - 2	≤ 6
M13M		Mid-Rise	3 - 5	6 - 15
M13H		High-Rise	6+	> 15
M2L	Adobe	Low-Rise	1 - 2	≤ 6
M31L	Wooden slabs URM	Low-Rise	1 - 2	≤ 6
M31M		Mid-Rise	3 - 5	6 - 15
M31H		High-Rise	6+	> 15
M32L	Masonry vaults URM	Low-Rise	1 - 2	≤ 6
M32M		Mid-Rise	3 - 5	6 - 15
M32H		High-Rise	6+	> 15
M33L	Composite slabs URM	Low-Rise	1 - 2	≤ 6
M33M		Mid-Rise	3 - 5	6 - 15
M33H		High-Rise	6+	> 15
M34L	RC slabs URM	Low-Rise	1 - 2	≤ 6
M34M		Mid-Rise	3 - 5	6 - 15
M34H		High-Rise	6+	> 15
M4L	Reinforced or confined masonry	Low-Rise	1 - 2	≤ 6
M4M		Mid-Rise	3 - 5	6 - 15
M4H		High-Rise	6+	> 15
M5L	Overall strengthened masonry	Low-Rise	1 - 2	≤ 6
M5M		Mid-Rise	3 - 5	6 - 15
M5H		High-Rise	6+	> 15

III.4.2.2 BTM and final vulnerability index

In terms of apparent damage, the seismic behavior of buildings is subdivided into vulnerability classes meaning that different types of buildings may behave in a similar way. The correspondence between the vulnerability classes and the building typology is probabilistic: each type of structure is characterized by prevailing (most likely) vulnerability class with the possible and less probable ranges (Giovinazzi and Lagomarsino 2004). Table III.6 presents the Building Typology Matrix (BTM) for buildings in Europe, which consists of 10 masonry building typologies. These typologies are shown with the respective attribution of vulnerability class according to the EMS-98 scale (Grünthal 1998).

Table III.6: Attribution of vulnerability classes to different building typologies



Typologies		Building type	Vulnerability Classes A	B	C	D	E	F
	M1	Rubble stone						
	M2	Adobe (earth bricks)						
	M3	Simple stone						
	M4	Massive stone						
	M5	Unreinforced M (old bricks)						
	M6	Unreinforced M with r.c. floors						
	M7	Reinforced or confined masonry						

Situations: ■ Most probable class; ■ Possible class; ■ Unlikely class (exceptional cases)

As already stated, the RISK-UE BTM is used to estimate the seismic vulnerability of European buildings. The first level (LM1) is based on the model of vulnerability contained implicitly in the Macroseismic scale EMS-98 (Grünthal 1998) and whose translation in quantitative terms has been created by Lagomarsino and Giovinazzi (2006). This led to the determination of the bases indices of building types with the definition of an average index V_0, with a limited range between V^- and V^+ values, which represent the possible variation of the final vulnerability index. Additionally, two extreme values that represent the upper and lower maximum V^{max}, V^{min} respectively are defined (Table III.7).

Table III.7: Vulnerability index values for each vulnerability class

Class	V^{min}	V^-	V^*	V^+	V^{max}
A	0.78	0.86	0.9	0.94	1.02
B	0.62	0.7	0.74	0.78	0.86
C	0.46	0.54	0.58	0.62	0.7
D	0.3	0.38	0.42	0.46	0.54
E	0.14	0.22	0.26	0.3	0.38
F	0.02	0.06	0.1	0.14	0.22

Therefore, from the both previous tables, the corresponding vulnerability index values for each masonry building typology is shown in Table III.8

Table III.8: Vulnerability indices for BTM masonry buildings (RISK-UE 2003)

Typology	*Description*	***Representative value of V*** V^{min}	V^-	V^*	V^+	V^{max}
M1.1	Rubble stone	0.62	0.81	0.873	0.98	1.02
M1.2	Simple stone	0.46	0.65	0.74	0.83	1.02
M1.3	Massive stone	0.3	0.49	0.616	0.793	0.86
M2	Adobe	0.62	0.687	0.84	0.98	1.02
M3.1	Wooden slabs	0.46	0.65	0.74	0.83	1.02
M3.2	Masonry vaults	0.46	0.65	0.776	0.953	1.02
M3.3	Composite steel and masonry slabs	0.46	0.527	0.704	0.83	1.02

M3.4	Reinforced concrete slabs	0.30	0.49	0.616	0.793	0.86
M4	Reinforced or confined masonry vaults	0.14	0.33	0.451	0.633	0.7
M5	Overall strengthened	0.30	0.49	0.694	0.953	1.02

III.4.2.3 Behavior Modifier factor ΔVm

There are different methods that evaluate the vulnerability through the weighted average or the sum of the partial scores to obtain a global score, which practically represents a vulnerability index (see Ghislaine (2008)). The RISK-UE LM1 is conceptually similar introducing the behavior modifiers. Table III.9 presents these parameters with the corresponding weights value for our case of interest of masonry buildings.

The overall score that modifies the characteristic vulnerability index V^* can be evaluated, for a single building, simply summing all the modifier scores (Eq. III.5).

$$\Delta V_m = \sum V_m \tag{III.5}$$

Table III.9: Scores for the vulnerability factors V_m: masonry buildings (RISK-UE 2003)

Vulnerability factors	Parameters	V_m
State of preservation	Good	-0.04
	Bad	+0.04
Number of floors	Low (1 or 2) Medium (3, 4 or 5) High (6 or more)	-0.02 0.02 +0.06
Soft-story	Demolition/transparency	+0.04
Plan irregularity		+0.04
Vertical irregularity		+0.02
Roof	Roof weight + roof thrust + roof connections	+0.04
Retrofitting interventions		-0.08 ÷ +0.08
Aggregate effect: building position	Middle Corner Header	-0.04 +0.04 +0.06
Aggregate effect: building elevation	Staggered floors Buildings of different height	+0.02 -0.04 ÷ +0.04
Soil Morphology	Slope Cliff	+0.02 +0.04

III.4.2.4 Regional Vulnerability Factor ΔV_R

A regional vulnerability factor ΔV_R is introduced to take into account the particular quality of some building types at a regional level. It modifies the vulnerability index V^* on a base of an expert judgment or taking into consideration of observed vulnerability. The regional vulnerability factor ΔV_R could be introduced to refer a typology or a category.

III.4.2.5 Total vulnerability index

The total vulnerability index value for a single building is calculated as follows (Eq. III.6):

$$V = V^{*} + \Delta V_{m} + \Delta V_{R} \tag{III.6}$$

It is worth noting that the final vulnerability index *V* evaluated by means of the typological factor V^{*} and the behavior modifiers factor ΔV_m as well as the regional factor ΔV_R using Eq. (III.6) has to comply with the following possible range (Eq. III.7):

$$Max\left(V^{*};V^{\min}\right) \leq V \leq Min\left(V^{*};V^{\max}\right) \tag{III.7}$$

III.4.2.6 Estimation of the damage distribution

a. <u>Damage grade definition</u>

The Level 1 (LM1) method is largely based on correlation between the macroseismic intensity and the observed damage from past earthquakes. It is derived starting from the European Macroseismic Scale (EMS-98) – the modern macroseismic scale that implicitly includes a vulnerability model, although defined in an incomplete and qualitative way. The LM1 method is based on five non-null damage grades labelled as Slight, Moderate, Substantial to Heavy, Very Heavy, and Destruction (Grünthal 1998). In addition, a non-damage grade is also defined which refers to the no structural and non-structural damages are occurred (Table III.10).

Table III.10: Classification of damage grade according to RISK-UE LM1 method

Damage Grade	Damage Label	Description
0 (D0)	None	No damage
1 (D1)	Slight	Negligible to slight damage
2 (D2)	Moderate	Slight structural, moderate nonstructural
3 (D3)	Heavy	Moderate structural, heavy nonstructural
4 (D4)	Very heavy	Heavy structural, very heavy nonstructural
5 (D5)	Destruction	Very heavy structural, total or near total collapse

b. <u>Mean damage grade</u>

The LM1 method defines a mean semi-empirical vulnerability functions that correlate the mean damage grade μ_D with the intensity (Eq. III.8) (RISK-UE 2003).

$$\mu_D = 2.5\left[1+\tanh\left(\frac{I+6.25V-13.1}{Q}\right)\right] \tag{III.8}$$

Where *I* is the macroseismic intensity described according to the European Macroseismic Scale, EMS-98 (Grünthal 1998), *V* is the final vulnerability index, and *Q* is a ductility factor that determines the slope of the vulnerability function. Following the work of Vicente *et al.*

(2011), the value of Q adopted in this work was equal to 2.3, which was previously calibrated and validated by (Giovinazzi 2005) for masonry buildings. Finally, the limit bounds μ_D^{--}, μ_D^{-}, μ_D^{+}, and μ_D^{++} are calculated with Eq. (III.8) from V^{--}, V^{-}, V^{+}, and V^{++}, respectively. Note that, according to the five damage grades defined in the EMS-98 scale, the value of μ_D ranges between 0 (no damage) and 5 (severe damage or destruction).

c. *Damage distribution*

In order to evaluate the seismic vulnerability of buildings located in European regions and towns, the macroseismic method, which is based on the EMS-98 macroseismic scale (Grünthal 1998) is able to assess the damage distribution and the imprecise determination of the damage probability. The damage distribution shall be calculated using the beta probability density function and the beta cumulative density function as given in Eqs. (III.9) and (III.10) respectively (RISK-UE 2003).

$$\text{PDF: } \frac{\Gamma(t)}{\Gamma(r)\Gamma(t-r)}\frac{(x-a)^{r-1}(b-x)^{t-r-1}}{(b-a)^{t-1}} \qquad a \le x < b \tag{III.9}$$

$$\text{CDF: } P_\beta(x) = \int_a^x p_\beta(\varepsilon)\,d\varepsilon \tag{III.10}$$

Where *a, b*, *t* and *r* are the parameters of the distribution, and *x* is the continuous variable which ranges between *a* and *b*. The parameters of the beta distribution are correlated with the mean damage grade μ_D as follows (RISK-UE 2003):

$$r = t\left(0.007\mu_D^3 - 0.052\mu_D^2 + 0.2875\mu_D\right) \tag{III.11}$$

The parameter *t* affects the scatter of the distribution; and if $t = 8$ is used, the beta distribution looks very similar to the binomial distribution. Moreover, to use the beta distribution, it is necessary to make reference to the damage grade D_k, which is a discrete variable characterized by 5 damage grades plus the grade zero damage (absence of damage). It is advisable to assign value 0 to the parameter *a* and value 6 to the parameter *b* (Giovinazzi *et al.* 2003).

The discrete *beta* density probability function is calculated from the probabilities associated with damage grades *k* and *k*+1 (*k* = 0, 1, 2, 3, 4 and 5), as follows (Eq. III.12) (RISK-UE 2003):

$$p_k = P_\beta(k+1) - P_\beta(k) \tag{III.12}$$

The fragility curve defining the probability of reaching or exceeding certain damage grade can obtain directly from the cumulative probability beta distribution as follows (Eq. III.13) (RISK-UE 2003):

$$P(D \ge D_k) = 1 - P_\beta(k) \tag{III.13}$$

III.4.3 Correspondence between both vulnerability indexes I_v and V

As the final vulnerability index is obtained differently in each method, a comparison between the GNDT II level approach and the macroseismic method (Table III.11) is made by Vicente *et al.* (2008).

Table III.11: Correlation between the vulnerability indexes and the vulnerability classes defined in terms of the EMS-98 scale (Vicente *et al*. 2008)

Macroseismic method	Class A (V = 0.88)	Class B (V = 0.72)	Class C (V = 0.56)
GNDT II level	$I_v = 50$	$I_v = 25$	$I_v = 0$

Based on the equivalence presented above, the following analytical correlation is derived between the vulnerability indexes indices of the two methods (Eq. III.14):

$$V = 0.56 + 0.0064 \times I_v \quad \text{(III.14)}$$

Via this relationship, the vulnerability index I_v, can be transformed into the vulnerability index V (used in the Macroseismic Method, RISK-UE), enabling the calculation of the mean damage grade through Eq. (III.14) and subsequently the estimation of damage and loss as will be shown in the chapter V.

Chapter IV: Exposure

IV.1 Introduction

Over recent years, seismic risk immensely increased in earthquake prone areas due to the rapidly growing spatial concentrations of people, infrastructure and financial values. Indeed, in a seismic densely populated area, which results in many deaths and considerable damage, may have the same magnitude as a shock in a remote area that does nothing more than frighten the wildlife. Large-magnitude earthquakes that occur beneath the oceans may not even be felt by humans. Therefore, an exposure analysis is performed by considering the built environment, the demographics and the environmental uses of the examined zone. In this sense, among the north Algerian cities, Annaba city should be addressed to a rigorous analysis of their exposures in fact that a rapid urban growth is accompanied by unplanned and highly vulnerable settlements, which dynamically change over short time-scales.

Therefore, the term exposure means the value of the exposed elements that may lead to a potential loss in a seismic event. Depending on the scope of the risk assessment study, exposure may include either a single building or a set of buildings with their occupants and contents or may include also all constructed facilities of the region (lifelines, and utility systems).

Building exposure information for a region requires a standard systematic inventory system that classifies the structures according to their type, occupancy, and function so that realistic estimates of seismic risk and loss can be made.

IV.2 Annaba city

Annaba (in Arabic; عنابة), its original name BOUNA; is a port city that stretches over an area of 34,900 km^2. Chief town of the wilaya of Annaba, is Annaba located at 152 km from the northeast of Constantine and 246 km east of Jijel, and about 80 km west of the Tunisian border (Fig. IV1). Annaba city has various sites combining the beaches to the mountains; but far from being limited its role to that of mere transit, it has greatly diversified by giving itself an industrial function and especially the animation tools in the economy of our vast country.

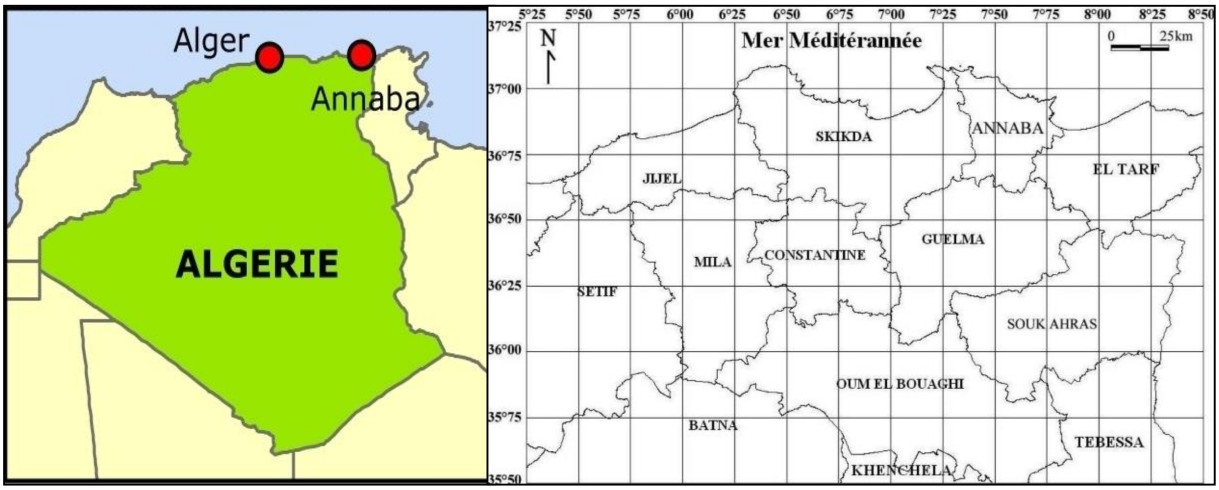


Fig. IV.1: Location of Annaba city

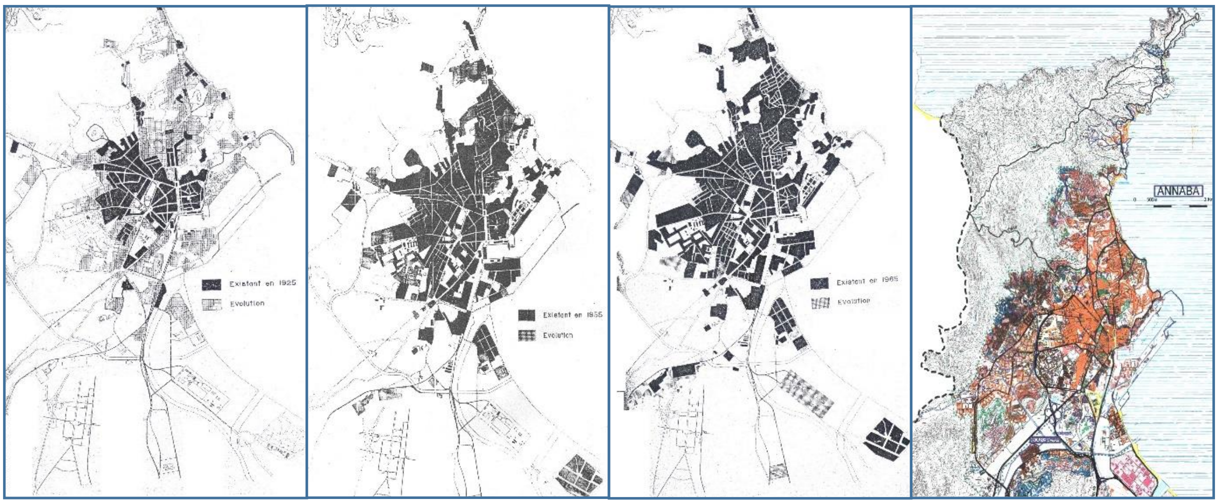
IV.2.1 History and urban evolution

Annaba is located in a geostrategic area, it has been throughout the centuries in contact with different civilizations. Therefore, the city and the region have experienced major events that have marked the history of Algeria, North Africa and the Mediterranean. Without going to the extensive details, the urbanization process of Annaba city is presented in Fig IV.2. During its evolution, we will interest by the French colonial era where many modifications were done on the buildings stock have been registered. In this regard, the urbanization began firstly by intervening in the medina in 1843 there the first work was on the "Cours Bertagna". Then the urbanization took place on the west and north-west of the Cours (neighborhoods: Champs de Mars, la Colonne Rondon, L'Elisa). After razing the hill of Santon, the urbanization was oriented to the north coast (neighborhoods: Beau-Séjour, Ménadia, Saint-Cloud).

- 1833: The places were opened, straightened and enlarged streets
- 1842: Construction begins on the road Edough
- 1843: Construction of the first buildings of the Cours BERTAGNA
- 1847: Completion of the construction of the church Saint Monique
- 1889: Construction of the suspension bridge connecting the old city and Caroubier
- 1909 -1928: The Boulevard along the Beau-Séjour and Ménadia was inaugurated.
- 1945: Abu Marouene returned MOSQUE.
- 1888: Construction of the City Hall
- 1909: Construction of the consular palace (now Chamber of Commerce).

After the independence, and with the growing need of housing, urban development policy has been oriented towards the realization of ZHUN (Habitat New Urban Zone), west and south of the city (Full West, El Bouni Sidi Amar, ...). Also, there has been the creation of public and private housing estates, and encouragement of the private sector in the field of construction.

1832-1849 1849-1868 1865-1905 1905-1925



1925-1955 1955-1965 1965-1990 1990-Up till now

Fig. IV.2: Urbane evolution of Annaba city (DUC 2006)

IV.2.2 Population and buildings stock

In term of population, Annaba city is the fourth largest city of Algeria after the capital Algiers, Oran and Constantine. Indeed, Annaba city is known by its dense population of 260,199 people, according to the 2011 census (DPBM 2011), where most of them are concentrated in the urban areas especially the old city center “Place d’arme”.

Regarding the building stock of the city of Annaba, a great variety of constructions is remarked, which is dominated nowadays by the reinforced concrete buildings. Nevertheless, a large number of buildings dates back to Ottoman and Colonial eras are still exist and represent the prevailing stock in certain areas of the city. The OPGI Annaba like other Offices of the great cities of Algeria, counts 62,480: 59,640 homes and 2,740 commercial. Considering the goal of this thesis, we should Note that 28,000 dwellings are classified as old buildings. Several studies and expertise have been conducted on this ancient tissue by the Technical Inspection Agency (CTC East). The latest expertise dated in February 2010, targeted 12 sites:

- Place d’arme (the study area)
- Beni M’haffeur
- Oued-Eddeheb
- Didouche Mourad
- La Colonne 1
- La Colonne 2
- Bélaid Belgacem
- Gazométre
- Centre-Ville
- Port Said
- Seybouse
- Sidi Brahim

IV.3 The old city centre of Annaba

The area of the old city of Annaba, better known by the common name of "Place d'Armes", once very large living center, located in the city center, which is bordered today as follows:

- in the North: the Boulevard du19 June
- in the East: the wall of the waterfront.
- in the South: the street from the Avant Port.
- in the West: CNRA street.

This urban area extends over a surface of 16 hectares on a Glaze overlooking the sea, which gives it a defensive character. Its slope that gradually decreases to the west gives it an opening towards the city. Its accessibility is mainly through the lower part. The upper part has a single access by a metal bridge (DUC 2006).

IV.3.1 History and urban evolution

According to the land use documents (DUC 2006), Place d'arme is one of the first settlement places in Annaba, started in the Arab-Turkish period, yet in the eighteenth century. Later, during the colonial French era (between 1830 and 1964), the town has been experienced a large expansion and modifications over the building blocks. Some of old buildings were replaced by colonial ones. Due to this fact, great part of the buildings in this urban area are a mixture of traditional and colonial structures (Fig. IV.3). It is important to stress that these buildings are representative of most historical urban areas in Algeria.

Fig. IV.3: Geography and aerial view of the old city center of Annaba (DUC 2006)

IV.3.2 Data inventory

An exposure assessment is achieved by means of an inventory of the elements at risk, consisting of a wide range of things, such as people and their economic activities, equipment, crops and livestock, the houses, the roads and the community services. Generally, these elements are not easily aggregated and have to be treated as a number of separate categories, having a different importance or value.

In the first two months of 2005, Annaba city witnessed tragic events in its historical centers: 49 collapse episodes were recorded — the most tragic of which was responsible for the death of an entire family due to the slump of an old building (CTC 2010), located in the old town. Consequently, in response to the frequent complaints of the inhabitants, the Direction of Urban Planning and Construction of Habitation (DUCH) launched a general program aimed at evaluating the vulnerability of the old buildings in 12 (out of 29) districts of the Annaba municipality, which have been declared as historical and heritage areas.

The data of the buildings stock were collected by a team of expert structural engineers of the CTC (the official technical organization of Annaba city in charge of the Technical Control of Construction) (CTC 2010). As already mentioned, the main scope of this survey was the evaluation of the degradation state of the individual buildings and the need for future interventions. In order to assess the degradation state of the buildings, the CTC experts adopted a qualitative approach based on a macroscopic inspection (Senouci *et al.* 2013). All the surveyed buildings were photographed and the photos were subsequently used to evaluate the information that was collected and registered in inspection protocols regarding both the interior and the exterior of each building.

An example of the CTC’s datasheets is presented in Fig. IV.4, which outlines the type of information collected during the field survey, subdivided in three parts: 1. Description; 2. Diagnosis; and 3. Conclusion and Recommendations. As can be shown in Fig. IV.4, the CTC data include valuable information that was used to classify buildings in one of the four state of health classes (good state, slightly degraded, moderately degraded, and highly degraded) in order to support different types of intervention purposes (repairs, strengthening, reconstruction, etc.) (CTC 2010). Each health class is assigned a color (green, yellow, orange, and red, respectively). The first group, in green, consists of buildings that do not require any structural intervention (except some possible minor repairs) to ensure the safety of their inhabitants. The second group, in yellow, consists of buildings which need repair and/or slight strengthening interventions. The third group, in orange, consists of buildings which require repair and/or serious rehabilitation intervention (moderate to heavy). The last group, in red, is consists of buildings that need urgent strengthening intervention or that should be demolished and reconstructed (CTC 2010).

Although the data was not originally developed for seismic purposes, such special engineering expertise on structural vulnerability is valuable and of great importance to obtain valid risk outputs.

As already mentioned, Fig. IV.4 presents the details of the parameters that listed in the CTC data sheets, to give an overview of the type of the items taken by the screeners in the diagnosis process.

Part I: DESCRIPTION

GENERAL INFORMATION

Use: Dwelling; Administrative; Commercial; ....
Year of construction: - Before 1832
- 1832 – 1962
- 1962 – 1980
- After 1980

Interventions on the construction: - None
- Aggrandizement
- Elevation
- Repair
- Rehabilitation
- Strengthening

Occupation: - Totally occupied
- Partially occupied
- Unoccupied
- Abandoned

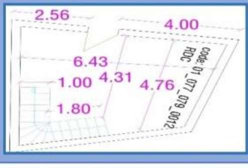
Implementation data: Type of ground on the surface; ....
General slope: - Nulle
- Moderate < 10%
- High > 10%

Aggregate position: ....
Diverse networks: ....

GEOMETRICAL FEATURES

Plan shape: ....
Elevation shape: - Number of floors,
- Height level
- Height above ground

Sketch:



BEARING STRUCTURE:

Vertical structure

Reinforced Concrete:	*Masonry*:	*Metallic structural*:
- Columns - Beams system	- Massive stone walls	- Ductile frame
- Frame system	- Rubble stone walls	- Ordinary frame
- Bearing walls system	- Adobe walls	- Triangular bracing in X, V
- Mixed system	- Tuff block walls	- Frame and bracing in X, V
- Ring system	- Mixed structure	- Assemblages
- Other:	- Other:	- Other : ....

Horizontal structure

- RC Slabs
- Slab with structure and wooden secondary elements
- Wood floors connected with steel ties
- Composite floor steel beams /masonry
- Masonry vaults with steel ties
- Presence of balconies and loggias
- Other: ....

Other description of : Partitioning and fillings; Stairs: Stairs type and structure; Guard Corps; Coating materials; Exterior siding; Blankets; Cladding; Waterproofing; carpentry / Joinery; Other installation; ....

Part II: DIAGNOSIS

BEARING STRUCTURE:

Vertical structure

Reinforced concrete:	*Masonry*:	*Metallic structural*:
- Supporting elements intact	- Subsidence	- No degradation
- Disappearance or insufficient of coating	- Bursting	- Bending
- Corrosion of Rebar and loss of section	- Fault of verticality	- Buckling
- Excessive deflection	- Misalignment	- Spillage
- Breaking	- Chalking	- Torsion
- Crushing	- Cracking	- Corrosion
- Distortion / Buckling	- Local collapse	- Rupture
- Cracks	- Dislocation	- Breaking
- Other:	- Other:	- Other: ....

Horizontal structure

- No degradation
- Undulation
- Subsidence
- Buckling
- Bending
- Bulge
- Corrosion
- Breaking
- Deterioration
- Other: ....

Degradation and deterioration stat of: Partitioning and fillings; Stairs; Covers ; cladding ; Waterproofing; ....
Immediate environment of the construction: ....
Appearance of traces of moisture: ....
Analysis: ....

Part III : CONCLUSION AND RECOMMENDATIONS

Degradation level of buildings: - Good state
- Slightly degraded
- Moderately degraded
- Highly degraded

Classification of the intervention: - Slight
- Moderate
- Heavy

Type of intervention: - Building to repair
- Building to strengthen
- Building to rehabilitate
- Building to demolish

Part of Photos:

Fig. IV.4: Parameters that compose the CTC data sheets (CTC 2010)

IV.3.3 Population and building stock

The majority of the inhabitants of Annaba city are concentrated in the old towns. The most interesting historical area of the city, which is usually called “Place d’arme”, shelters about 12,000 people (DUC 2006). Moreover, the built-up area is dominated by the multi-family residential buildings, which are usually fully occupied (Fig. IV.5) (CTC 2010).

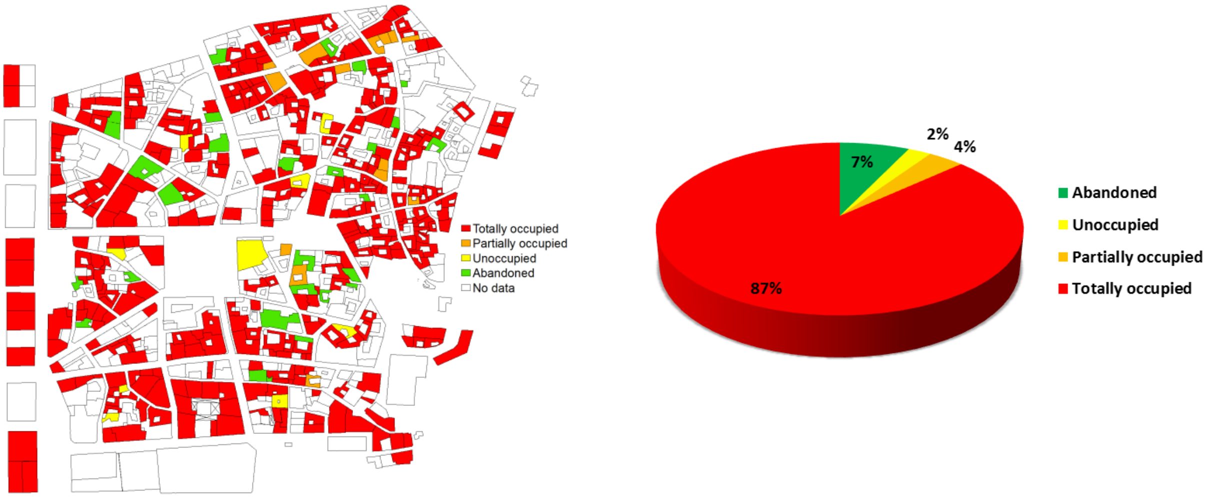


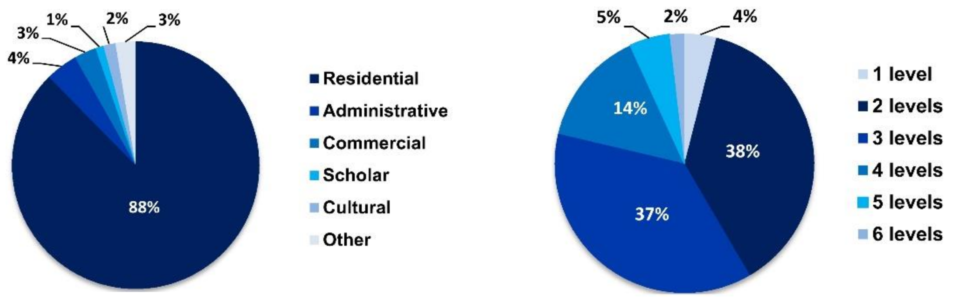
Fig. IV.5: Spatial distribution of the collected data according to habitability state, after (CTC 2010)

This old historical center is composed only of old masonry structures built making narrow alleys and streets (Fig. IV.6).

Fig. IV.6: Alleys and streets of the study area “Place d’arme” (DUC 2006)

The CTC data used for the working area selected in this thesis, involve 380 masonry buildings of the old town out of 602 buildings for which the data were collected in 2010 (CTC 2010). The results of the inspection have shown that the buildings are being used for different purpose, either administrative, scholar, cultural or others, such as military and production centres (Fig. IV.7a). Often, the ground storey of a considerable number of those buildings is used for commercial ends. It is worth noting that this study does not take into account the monumental, the scholar and the cultural buildings, which must subject to specific analysis.

Regarding the class of height, the constructions of the Place d'arme marked a variety of floor levels such as: GF, G+1, G+2, G+3, G+4 and those of G+5 (Fig. IV.7b). The most dominant levels are those of G+2, G+3 with an absolute value of 207, 198 (38% and 37% respectively). This dominance is remarkable in the buildings which were not undergo many changes and transformation, they almost have kept the same Turkish design (spatial organization and height). Furthermore, a small number of buildings of G+5 in height is exist, which is explained by the topographic constraints and the influence of the Turkish urban organization (shape, plot and height). These tallest buildings are located in the lower part of the old town, this concentration is due to the massive intervention of the French, since almost all of these buildings were either transformed or completely renovated while emphasizing on height. The buildings of the same type are generally well distributed, indeed, the tallest buildings still remain in band, otherwise a large majority of low houses are in great together.



(a) Usage (b) Floor

Fig. IV.7: Distribution of buildings stock according to: a) usage and b) N° of floors, after (CTC 2010)

Over time the vulnerability of this old center has been increased, mainly due to the very poor conditions of the building stock, which reveals high levels of interior and exterior degradation (Fig. IV.8) (CTC 2010).

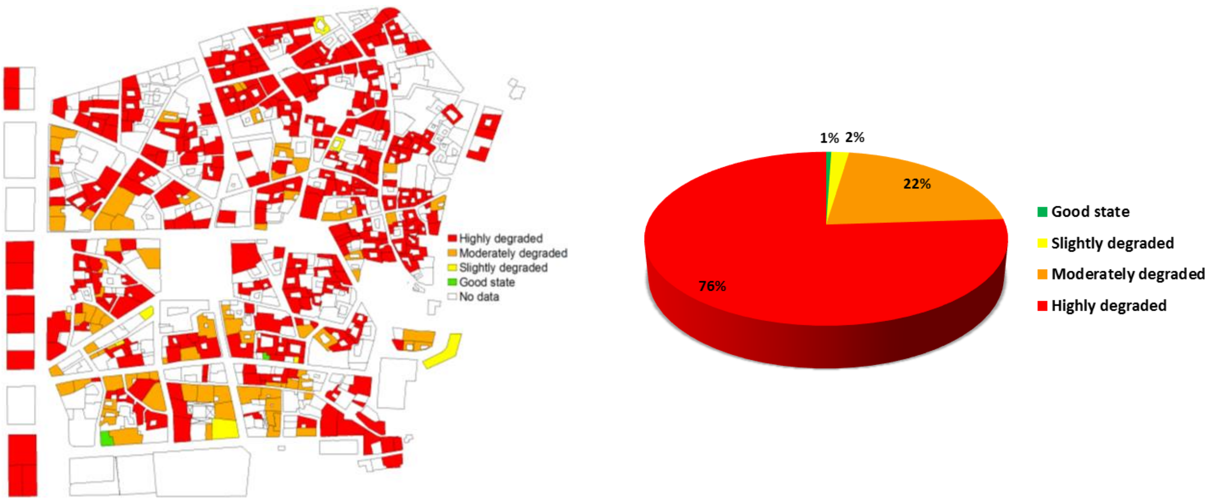


Fig. IV.8: Spatial distribution of the collected data according to health state

This situation is compounded by the poor quality of the construction materials used. Moreover, the condition and the connections between the various structural elements are often insufficient (Lazzali and Bedaoui 2012) as can be deduced from the figures below (CTC 2010).


1

2

3

8

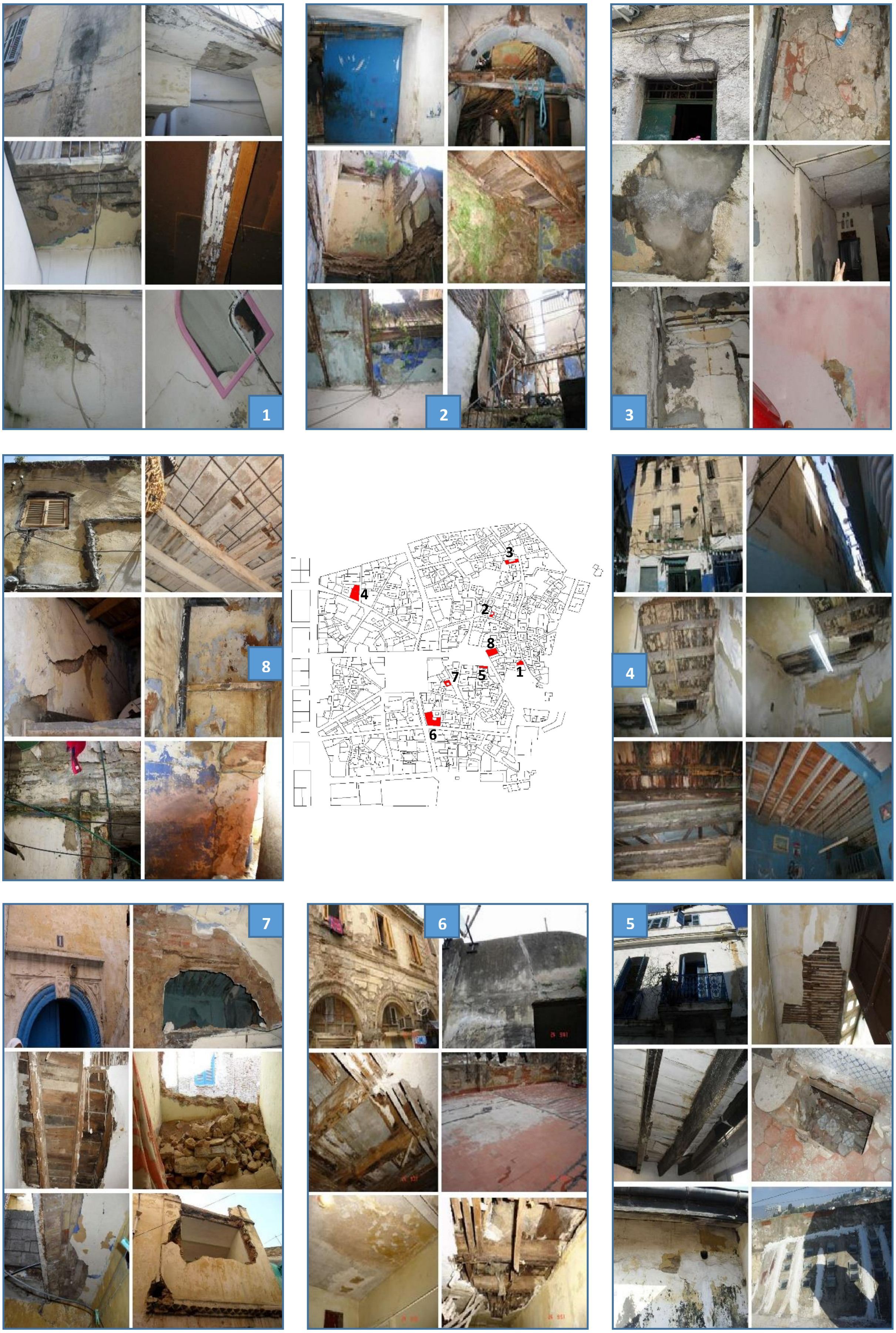
8
3
4
7
5
6
1
2

4

7

6

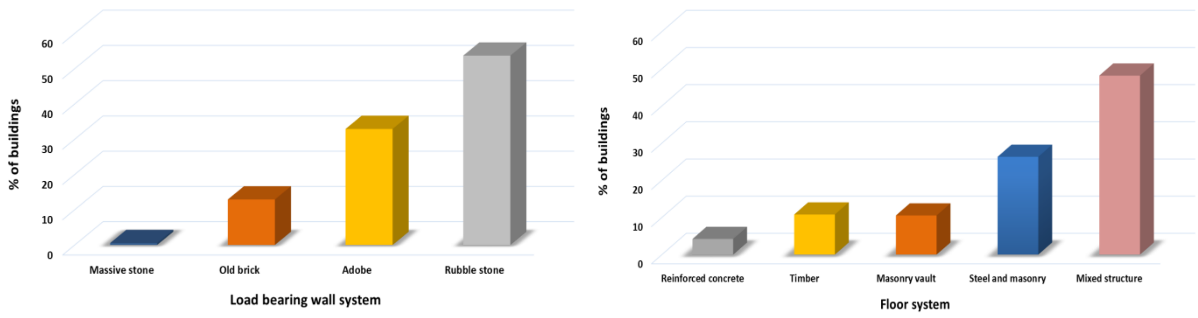
5


1
2
3
4
5
6
7
8


1
2
3
10
4
9
5
8
7
6

Concerning the material constitution of the resisting system, stone, fire clay bricks and adobe are widely used for the residential buildings in the old town of Annaba. As it can be concluded from Fig. IV.9a, two categories clearly dominate: rubble stone and adobe of about 54% and 33%, respectively. Unfortunately, these materials reflect the most vulnerable typologies to seismic events according to the EMS-98 scale (Grünthal 1998), therefore requiring particular attention (Santos *et al.* 2013).

In the other hand, some of these structures present a regular structural layout with thick walls (usually ranging from 40 to 50 cm), containing a commercial ground floor and residential apartments above. The ground floors, which are frequently in very bad condition (CTC 2010), are usually made of timber or a mixed system of steel and brick masonry vaults (Fig. IV.9b). Timber floor constructions include wooden beams covered with wooden planks, ballast fill, and tile flooring. In the CTC data, the floor of buildings of two or more types is marked as mixed structure. Moreover, reinforced concrete and stone or brick vaults floors are also observed in the old city center of Annaba (Fig. IV.9b).



a) load bearing wall system b) floor system

Fig. IV.9: Distribution of buildings according to structural systems, after (CTC 2010)

Regarding the geological characteristics of Annaba city, the underlying ground consists of three main geological rock units: the upper part of the town is characterized by “Gneiss”, whereas the majority of the area from the middle to the northwest part is above “Micaschistes” and “Cipolins” layers. Over the whole study area, the soil conditions can be considered stiff to very stiff, presenting an average bearing capacity of 200 KPa (DUC 2006). Therefore, it was assumed in the present study that the spatial distribution of damage should not be significantly affected by site conditions, but mainly reflects the buildings vulnerability.

Chapter V: Contribution

V.1 Introduction

In operative terms and in respect to the definition of seismic risk provided in (Eq. I.1), a vulnerability method must correlate the seismic hazard evaluation to the physical damage suffered by the built system depending on the structural, geometric, technological characteristics able to affect the seismic building behavior (Giovinazzi 2005).

To briefly summarize the previous chapters, it should be noted that intensity-based procedures rely on statistics data are thus more reliable in terms of buildings vulnerability of our case study. This applies especially to those building that show large variations in typology and are thus more problematic to model analytically. But these studies are more subjective with respect to the description of the hazard. On the other hand, the analytical (capacity spectrum-based) approach is more objective in terms of defining the seismic hazard as it considers physical measures of seismic ground motion and is at best based on instrumental recordings (Lang 2012). As already mentioned, building vulnerability processed by the analytical models need to be calibrated using damage statistics (hybrid methods). However, in the absence of this calibration, seismic risk evaluation derived by analytical approaches may not be better than intensity-based results. It can therefore be concluded that the analytical approach should be preferred in cases when reliably calibrated vulnerability models are available.

V.2 The applied methodologies

As already stated, two methods have been applied in this paper: the GNDT level II approach, which is assumed here as the main method used to assess the seismic vulnerability of the built-up area under study; and the RISK-UE LM1 approach, which was selected on the basis of what is exposed in the CETE (2008) report. This latter method is considered to be the best one used for the first level of seismic assessment study, due to the quality and reliability of the results obtained from it and the level of scientific validation.

V.2.1 GNDT approach

Since this early phase, this method has been used worldwide with several modifications (Calvi and Pinho 2006, Giovinazzi and Lagomarsino 2004, Guéguen *et al.* 2007, Lagomarsino and Magenes 2009, McGuire (2004), Neves *et al.* 2012, NZSEE 2006, RISK-UE 2003, Srikanth *et al.* 2010, Vicente *et al.* 2008). In Algeria, numerous recent studies have been oriented to the adaptation of these vulnerability index methods (Bensaibi *et al.* 2003, Bensaibi *et al.* 2011, Boukri and Bensaibi 2006, 2008, Djaalali and Bensaibi 2009, Djaalali *et al.* 2012) especially for masonry constructions.

In the same way, the applied method comes as an adaptation of the original GNDT II approach for the masonry buildings of Annaba city, having been improved and simplified by (i) giving a fixed vulnerability class of certain parameters to all types of construction defined on the basis of the general characteristics of the built environment of Annaba city; (ii) clarifying the definition of the most important parameters by means of their adaption to the CTC data; and

(iii) introducing new parameters that take into account the overlooked building features of the built environment of Annaba city.

The assessment based on the GNDT approach is proposed in this work using a seismic vulnerability index, which results from the weighted sum of 14 different parameters (listed in Table V.1) using the Eq. (V.1).

$$I_v^* = \sum_{i=1}^{14} C_{vi} \times P_i \qquad \text{(V.1)}$$

Where C_{vi} is the vulnerability class and P_i is the weight associated to the corresponding parameter. As shown in Table V.1, the weight P_i assigned to each parameter is practically the same taken in the original method (GNDT II). However, the weight of the additional parameters (P1, P6, P8, and P12) is defined on the basis of previous studies and from post-earthquake damage data collected following past earthquakes in Algeria. For instance:

- Regarding the masonry buildings located in Annaba city and especially in the area under study, the main bearing wall elements were built with the worst typologies (Fig. IV.9) of low quality of material (CTC 2010), which are usually considered as the most vulnerable to the seismic events (Grünthal 1998). Indeed, based on the statistical study of the damages performed after the well-known earthquake of May 21, 2003 in Boumerdès city (Algeria) (CGS 2003), the results showed that the masonry buildings suffered mainly heavy damages to collapse. From these observations, a weight of 2.5 is proposed for the typology of the resisting system parameter (P1), which is a value equivalent to that used by VULNERALP 2.0 (Guéguen *et al.* 2007), which was already applied in the Algerian urban context (Senouci *et al.* 2013).

- As already mentioned, the number of floors for the buildings located in the old town of Annaba city range between two and six, with most buildings being two or three stories. According to several past studies (e.g. Vicente et al. 2011; Ferreira et al. 2013), in general the seismic vulnerability of masonry buildings is directly related with their height — i.e. tall buildings tend to be more vulnerable than low buildings. Furthermore, in our case study, the number of dwellings contained into the building was also taken into account, considering the rate of risk against a seismic event due to the high population density of the area (Fig. IV.5). Therefore, the proposed weight assigned to this parameter is 0.75.

- The old areas in Annaba city are known by their dense aggregates of buildings, which are composed of compact islets. In these aggregates, the buildings are frequently connected, and therefore isolated masonry structures are uncommon. In term of global stiffness, the seismic effect of such a situation is very significant, especially for the buildings located at the extremities and corners of the aggregates (Lefebvre 2004). Thus, a weight of 0.75 is assumed for parameter P8.

- The intervention parameter (P12) is used in accordance with what was applied by Boukri and Bensaïbi (2008) in the Algerian city of Algiers for the parameter "modifications" with the same weight of 0.5. This parameter assesses the anomalies of the construction that affects the structural behavior of buildings (for example, the additions or the suppressions of certain elements) causing a change in the center of mass, and therefore influences the value of the seismic response of the structure.

The final vulnerability index I_v^* is then normalised to fall within the range 0 and 100 in order to estimate the building damage under a specified seismic intensity in the same way of the GNDT "mother-method". It is worth highlighting that the method proposed herein is considered robust, taking into account the CTC data — where the majority of buildings in the area under study were analyzed in detail — and the availability of further accurate geometrical information. Therefore, uncertainty in the assignment of vulnerability classes to each parameter is considered acceptable for a large-scale seismic vulnerability evaluation. Table V.1 summarizes the 14 parameters used in the formulation of the vulnerability index, as well as the information used.

Table V.1: The modified GNDT II method

PARAMETERS		Vulnerability Class C_{vi}				GNDT weight	Proposed weight
		A	**B**	**C**	**D**	W_i	P_i
P1	Typology of resisting system	0	5	25	45	-	2.50
P2	Organization of the resisting system	0	5	25	45	1.00	1.00
P3	Conventional strength	0	5	25	45	1.50	1.50
P4	Maximum distance between walls	0	5	25	45	0.25	0.25
P5	Horizontal diaphragms	0	5	25	45	1.00	1.00
P6	Number of floors	0	5	25	45	-	0.75
P7	Location and soil conditions	0	5	25	45	0.75	0.75
P8	Aggregate position and interaction	0	5	25	45	-	0.75
P9	Plan regularity	0	5	25	45	0.50	0.50
P10	Vertical regularity	0	5	25	45	0.50	0.50
P11	Roof system	0	5	25	45	0.25	0.25
P12	Interventions	0	5	25	45	-	0.50
P13	General state of preservation	0	5	25	45	1.00	1.00
P14	Non-structural elements	0	5	25	45	0.25	0.25

Information available from CTC data survey
Information completed from other sources
Parameter adapted according to CTC data survey
Proposed a fix vulnerability class for all buildings

The 14 parameters are arranged into four groups assigned by different color to emphasize their possible evaluation according to the available data survey.

As shown in Table V.1, the first group (in green) includes parameters for which full information is available from the CTC data survey (CTC 2010). Since the available CTC data only provides partial information, the second group (in yellow) includes the parameters for which complementary information from other sources was used in their evaluation. The parameters highlighted in grey are assessed and adapted using only the information of the CTC data. The last group (in blue) refers to the only parameter that could not be inferred from the CTC data (P14).

For more deep details, the last chapter "Appendix" presents the original descriptions of the 14 parameters and their adaptation using especially the CTC data survey.

V.2.2 EMS-98 approach

Based on the original RISK-UE LM1 defined in Chapter III is applied herein to evaluate the vulnerability and the seismic risk of the historical buildings stock of the old town of Annaba city. However as already indicated some implicit and explicit modifications were done to accounting the context of the area under study. In this sense, based on the information for the types of vertical and horizontal load-bearing structures, the building typology was deduced according to the RISK-UE LM1 method (see Appendix). However, according to the current data, certain buildings present more than one masonry typology or horizontal structure. Hence, in such cases the RISK-UE classification was based on the best described element, usually corresponding to the oldest and consequently the most degraded (see Appendix).

The probable vulnerability index V^* computed for each typology can be increased or decreased on the basis of specific structural modifiers, ΔV_m, (Giovinazzi 2005) (see Chapter III). For the aim exposed above, in the current applied methodology, two additional parameters were added to take into account the preservation state of the load bearing wall system and the diaphragm system (see Appendix).

V.3 Vulnerability assessment and reliability of results, damage distributions and scenarios

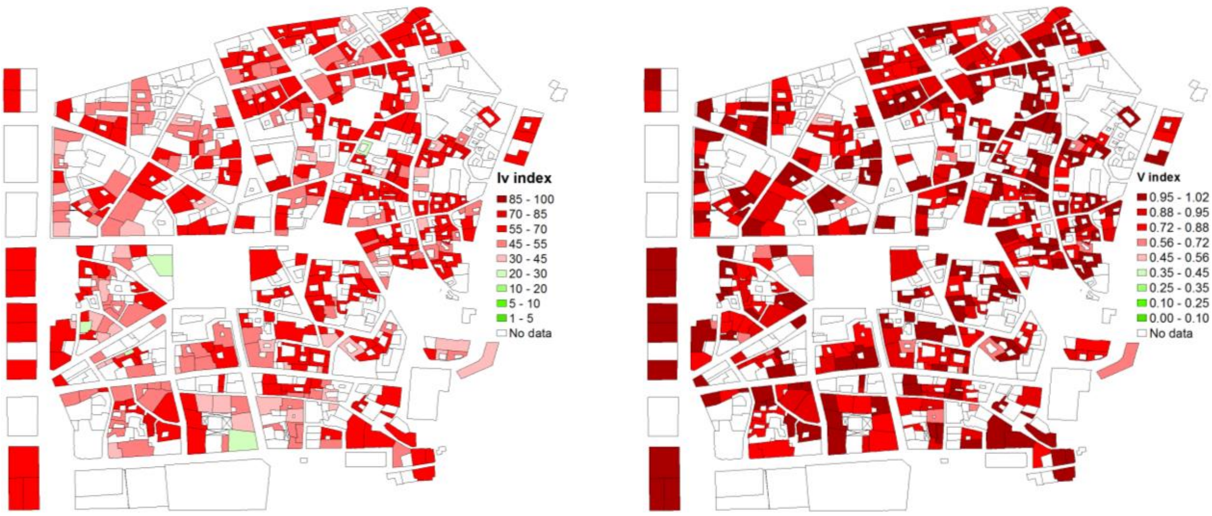
V.3.1 Seismic vulnerability assessment

As mentioned, the entire traditional masonry building stock of the old city center of Annaba was herein assessed. As a result, firstly a vulnerability index (I_v) value was assigned to each building deduced from the modified GNDT II method. A mean vulnerability index value of I_v = 57.86, with a maximum and minimum value of 78.98 and 23.67, was obtained for the whole building stock. Additionally, according to this methodology, almost 91% of the buildings have a vulnerability index value of over 45 (equivalent to vulnerability class A).

Regarding the RISK-UE LM1 method, the mean vulnerability index value V = 0.91 lies between the maximum and the minimum values of 1.02 and 0.57 respectively. Moreover, a similar rate of 90% of the assessed buildings fall into the vulnerability class A, with a vulnerability index V value higher than 0.78.

It is important to notice that, based on the correlation between the vulnerability indexes and the vulnerability classes defined in terms of the EMS-98 scale (Grünthal 1998) for the two applied methods, the corresponding maximum and minimum values were obtained for the building typology of massive masonry and rubble stone masonry, which correspond to vulnerability classes A and C, respectively.

Fig. V.1 shows the spatial distribution of the building stock using the seismic vulnerability indexes (I_v, V) derived from the two proposed methodologies. Such seismic vulnerability maps enable the identification of the most vulnerable buildings, constituting a valuable output for civil protection strategies.



a) **b)**

Fig. V.1: Vulnerability index according to: a) GNDT II and b) RISK-UE LM1

V.3.2 Validation of the reliability and robustness of results

The reliability of results is carried out herein through the comparison between the mean damage grades μ_D and D, worked out with the two applied methods RISK-UE LM1 and GNDT II, respectively, based on the estimation of residual damage value D_R computed according to Eq. (V.2) (Guéguen *et al.* 2007) for each one of the assessed buildings.

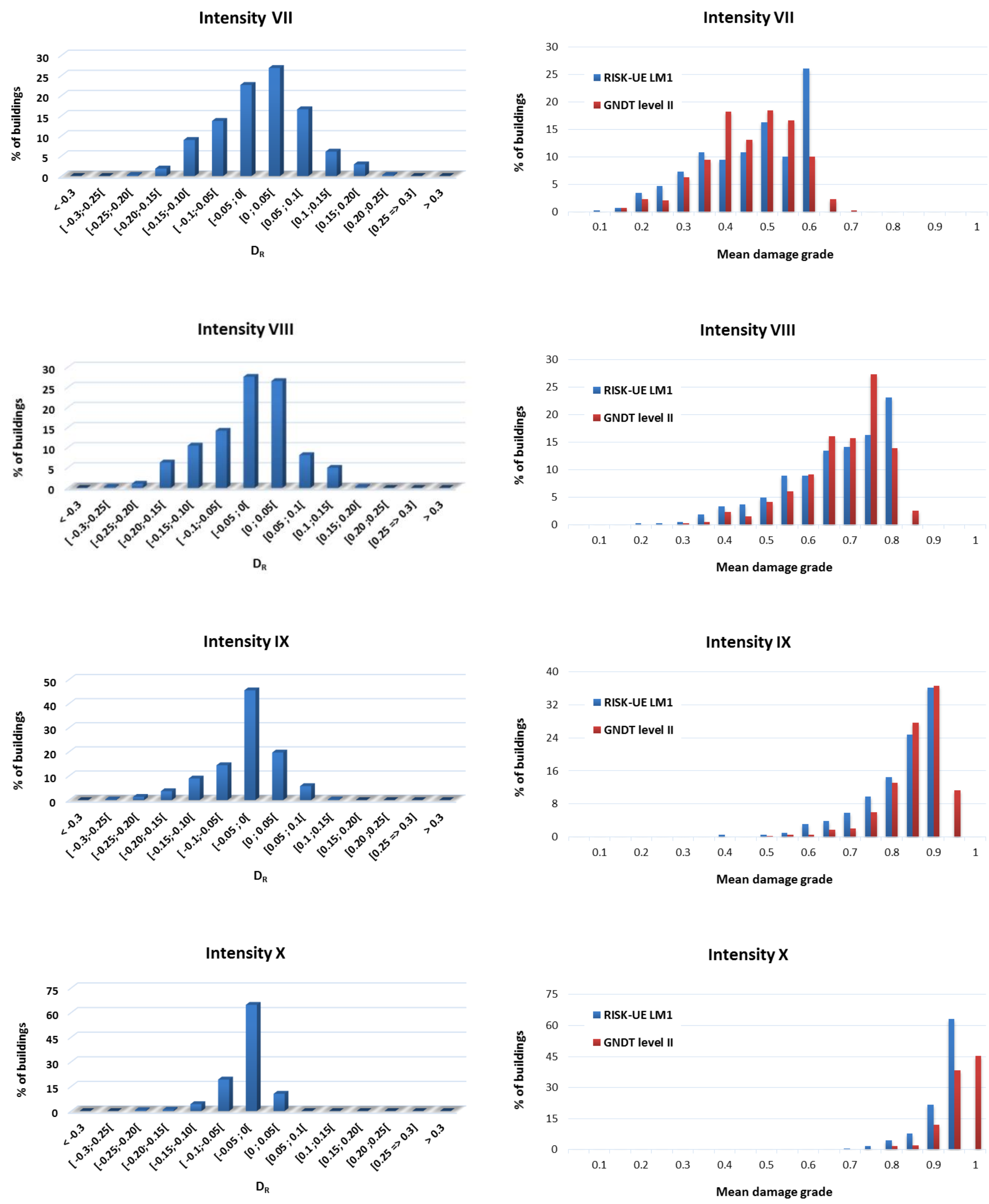
$$D_R = \frac{\mu_D}{5} - D \tag{V.2}$$

It is important to note that in Eq. (V.2) the mean damage grade μ_D is divided by 5 in order to normalize it from 0 to 1, as in GNDT d damage. This fact is due to the difference in definition and translation of the vulnerability index into damage for a given macroseismic intensity (Eqs. (III.4) and (III.8)) in the two selected methods. Consequently, the comparison is done only in terms of the mean damage d of the GNDT approach and μ_D of the RISK-UE method.

The comparison presented in Fig. V.2 shows that the two methods give similar results, where the residual value D_R for all intensities fall into a range of less than one damage grade (according to the EMS-98 scale), reflecting the good convergence between the two methods applied. For instance, for an intensity $I_{EMS\text{-}98}$=VII, the majority of the buildings present a residual value D_R ranging between -0.2 and +0.21, which corresponds to the same damage grade according to EMS-98.

For an intensity $I_{EMS\text{-}98}$=VIII, almost all buildings have a residual value less than one grade on the EMS-98 damage scale ($-0.2 \leq D_R \leq +0.2$). In fact, only 1% of the assessed buildings have a residual value corresponding to more than one damage grade. The minimum and maximum values obtained from this procedure were -0.27 and 0.17, respectively. Finally, for the intensity $I_{EMS\text{-}98}$=IX, the residual values of the great majority of the buildings range between -0.2 and +0.2. In this case, 2% of buildings present a residual value corresponding to more than one grade. The minimum and maximum values obtained were -0.28 and 0.10, respectively.

The consistency of results between the two applied methodologies is also implied by the results obtained for an intensity $I_{EMS\text{-}98}$=X, for which the extreme residual values D_R, were similar to the ones of the previous cases (-0.2 and +0.2). In this last case, the minimum and maximum values obtained from the assessment were -0.23 and 0.04, respectively.



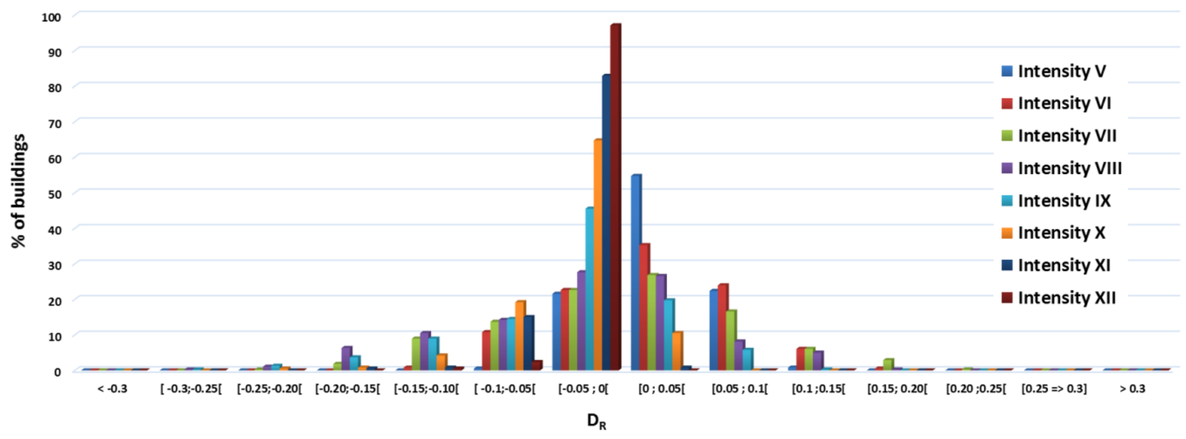


Fig. V.2: Comparison of the results derived from the two methods (RISK-UE LM1 and GNDT II) in terms of the residual damage value (D_R)

The results presented in Table V.2 show that, according to the EMS-98 scale and considering an interval less than one degree [-0.2; 0.2], the residual value D_R is almost linear for all the analyzed macroseismic intensities, ranging between 98% and 100%. As an example, considering intensities I_{EMS-98} of VII, VIII, and IX, respectively, 99%, 99%, and 98% of the Annaba old masonry buildings have a residual value lower than 0.2. As can be seen in Table 6, for the remaining macroseismic intensities, all the masonry buildings assessed presented a residual value within the range of -0.2 to +0.2.

Table V.2: Percentage distribution of D_R values for different EMS-98 intensities

	I_{EMS-98}	V	VI	VII	VIII	IX	X
	[-0.1; 0.1]	99	93	80	77	86	94
	[-0.15; 0.15]	100	99	95	92	95	99
D_R	**[-0.2; 0.2]**	**100**	**100**	**99**	**99**	**98**	**100**
	[-0.25; 0.25]	100	100	100	100	100	100
	[-0.3; 0.3]	100	100	100	100	100	100

The good agreement found between the two applied methods, presented above in Fig. V.2 and in Table V.2, is further depicted in Fig. V.3 by means of histograms showing the distribution of the mean damage values (expressed in EMS-98 grades) for the masonry buildings located in the old city center of Annaba.

As can be observed, an increase of one grade of intensity leads to an increase of one damage grade for both methods. Such a conclusion can be pointed out on the basis of the fact that the masonry buildings are well surveyed and detailed through the CTC database, which allowed a good application of the proposed modifications implemented on the two methodologies (RISK-UE LM1 and GNDT level II). In addition, it also explains the high vulnerability of the assessed buildings against the seismic events.

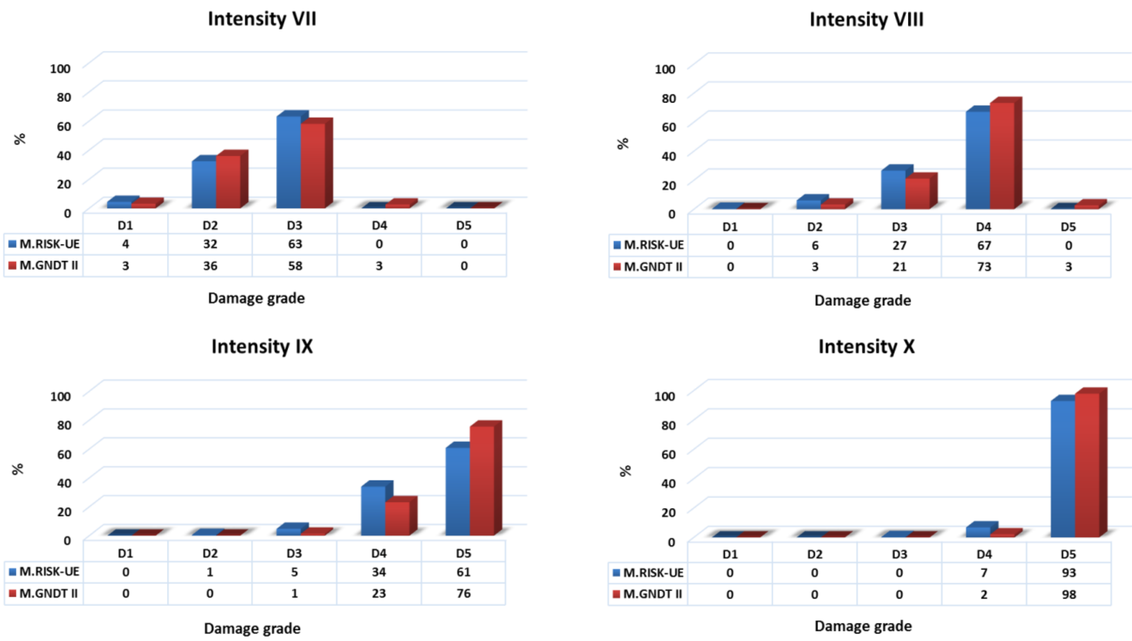


Fig. V.3: Distribution of the damage grades μ_D and D for the EMS-98 intensities VII, VIII, IX, and X. For the mean damage µD the different grades are defined as following: D1, 0–1 (negligible to slight, no structural damage, slight non-structural damage); D2, 1–2 (moderate, slight structural damage and/or moderate non-structural damage); D3, 2–3 (substantial to heavy, moderate structural damage and/or heavy non-structural damage); D4, 3–4 (very heavy, heavy structural damage and/or very heavy non-structural damage); D5, 4–5 (very heavy, total or near-total collapse)

V.3.3 Seismic scenario and distribution of physical damage

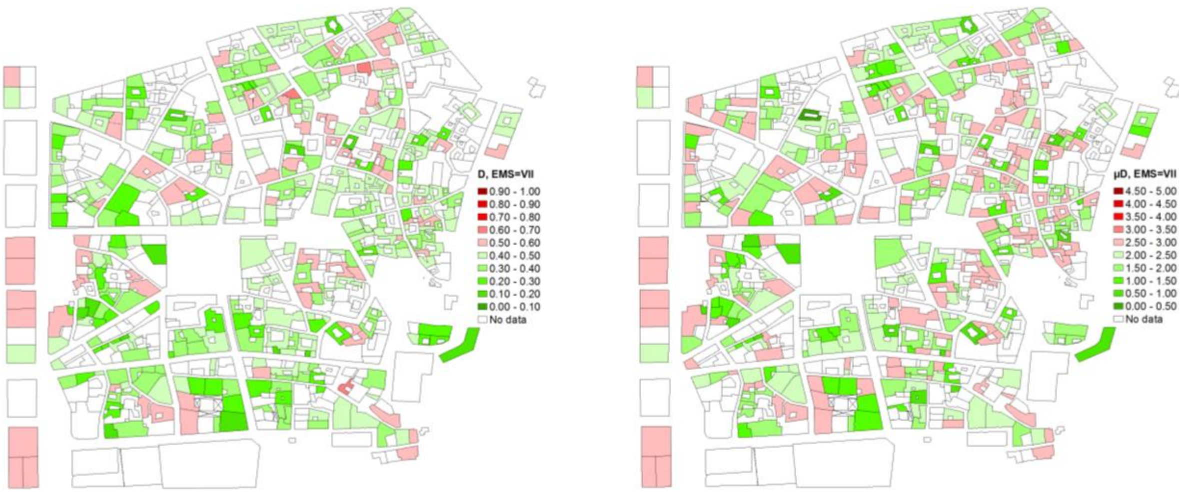
The damage assessment presented above was carried out using two methods and is shown as damage scenarios for earthquake intensities between $I_{EMS\text{-}98}=VII$ and $I_{EMS\text{-}98}=X$ (Figs. V.4, V.5, V.6, and V.7). Such seismic vulnerability maps enable damage appraisal, and are therefore very useful tools for the focused implementation of both individual or/and larger-scale urban retrofitting processes and strengthening strategies.

As can be seen in Fig. V.4, where a seismic scenario for the intensity $I_{EMS\text{-}98}=VII$ is presented, both maps are similar in terms of distribution of the damage grade (0 to 5) or (0 to 1) according to μ_D and D, respectively. Regarding the RISK-UE method, the obtained estimated damage ranges between 0.46 and 2.80. As shown in Fig. V.3, the majority of masonry buildings have a mean damage grade $1 < \mu_D \leq 3$, which refers to a probable damage between D_2 and D_3 (expressed in terms of the EMS-98 scale) with a rate of 32.37% and 63.16%, respectively. Regarding the modified GNDT II method, similar results were obtained. In this case, the minimum and the maximum values were 0.12 and 0.68, respectively. According to Fig. V.3, the mean damage for the majority of the buildings ranges between damage grade D_2 and D_3 ($0.2 < D \leq 0.6$) with a very similar percentage distribution with the ones obtained by the RISK-UE method, 36.05% and 58.16%, respectively.

Fig. V.5 shows that for intensity $I_{EMS\text{-}98}=VIII$, the values for the damage grades μ_D and D are very similar for the majority of the masonry buildings assessed. The expected damage grades computed with RISK-UE method range from D_3 to D_4 ($2 < \mu_D \leq 4$), wherein severe damages and potential local collapses are expected for about 27% and 67% of buildings, respectively. Only 6% of the analyzed buildings present a damage grade D_2. The minimum and the maximum values obtained for μ_D are 0.86 to 3.76. Considering the modified GNDT II method the pattern is quite similar, with mean damage values for the majority of buildings ranging between damage grades D_3 and D_4 ($0.4 < D \leq 0.8$) for 21.05% and 73.16% of buildings, respectively. Moreover, 2.63% of buildings are susceptible to collapse ($D > 0.8$). The minimum and the maximum values of the estimated damage were determined to be 0.27 and 0.84, respectively.

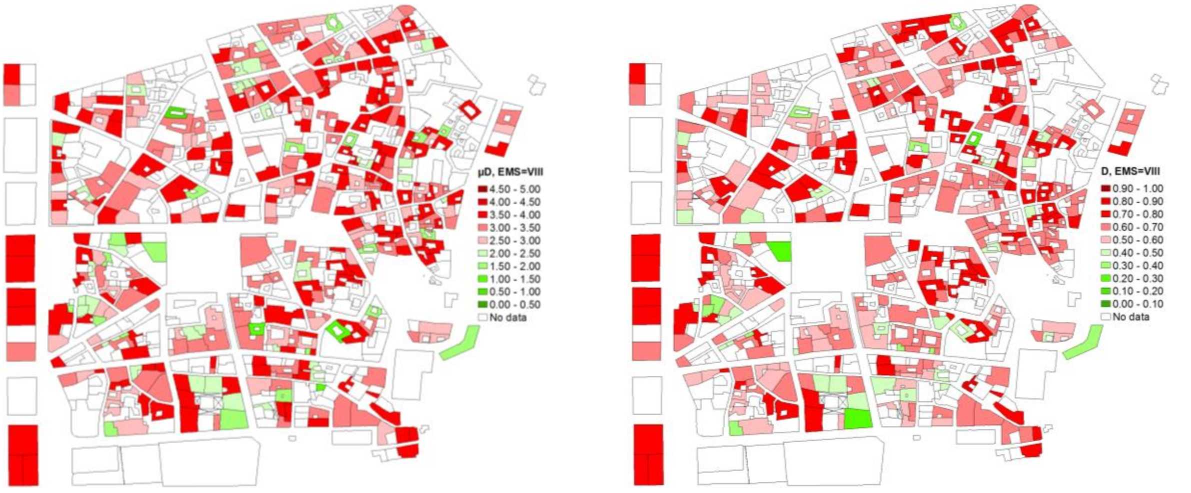
Fig. V.6 presents the damage patterns obtained considering a macroseismic intensity $I_{EMS-98}= IX$. It can be observed that the damage grade distributions are similar. For RISK-UE method, more than half of the total buildings (about 61%) exhibit susceptibility to collapse ($\mu_D>4$), corresponding to grade 5 according to EMS-98. Additionally, about 34% of the masonry buildings are resolved with a near collapse state, presenting D_4 ($\mu_D>3$). The minimum and the maximum estimated values of the mean damage grade obtained were 1.84 and 4.39, respectively. According to the second method (modified GNDT II), a similar pattern is observed, with almost a similar percentage of damage distribution — i.e. damage grades D_4 and D_5 ($0.6 < D \leq 1$); 23% and 76%, respectively. The minimum and the maximum values obtained for the estimated damage were 0.50 and 0.94, respectively.

Once again, similar results were obtained for the two methods considering a macroseismic intensity $I_{EMS-98}=X$. In this last scenario (Fig. V.7), the great majority of the masonry buildings have a mean damage grade ranging between D_4 and D_5 ($3 < \mu_D \leq 5$), which refers to a probable damage between D_4 and D_5 (expressed in terms of the EMS-98 scale). Considering such intensity, the majority of masonry buildings located in the old city center of Annaba should be collapsed at a percentage of about 7% and 93% for damage grades D4 and D5, respectively, according to the RISK-UE method. The values of 2.90 and 4.73 were the peak values (minimum and maximum) for this damage scenario. Regarding the modified GNDT II method, a similar pattern is obtained. In this case, about 2% and 98% of the masonry buildings present a damage grade ranging between D4 and D5, while the minimum and the maximum values are 0.72 and 1.00, respectively.



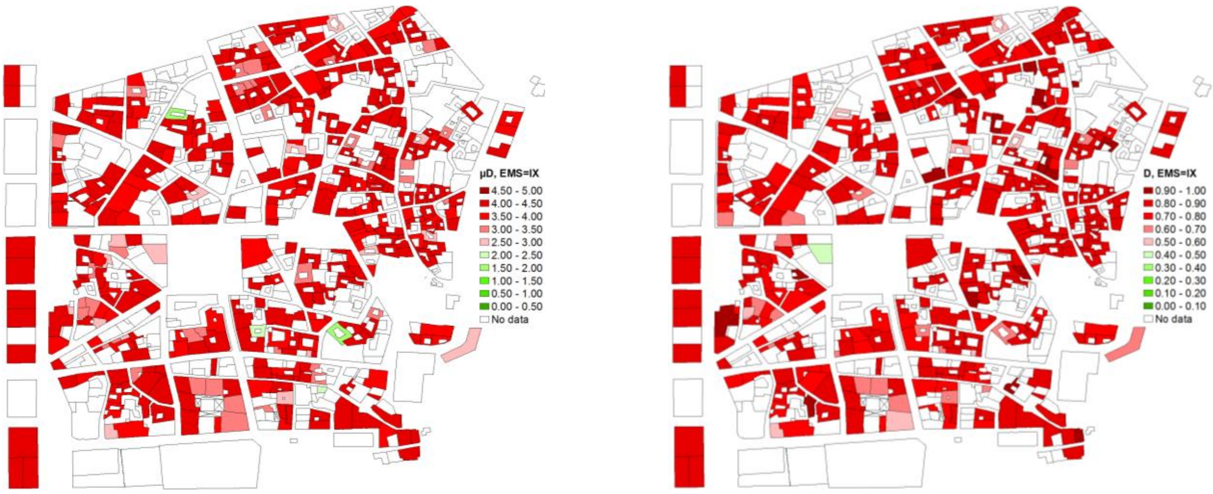
a) **b)**

Fig. V.4: Damage scenarios for I_{EMS-98} = VII obtained for the applied methods: a) RISK-UE and b) GNDT II



a) **b)**

Fig. V.5: Damage scenarios for $I_{EMS\text{-}98}$ =VIII obtained for the applied methods: a) RISK-UE and b) GNDT II



a) **b)**

Fig. V.6: Damage scenarios for $I_{EMS\text{-}98}$ = IX obtained for the applied methods: a) RISK-UE and b) GNDT II

a) **b)**

Fig. V.7: Damage scenarios for $I_{EMS\text{-}98}$ =X obtained for the applied methods: a) RISK-UE and b) GNDT II

V.3.4 Mean vulnerability curves

Vulnerability curves are another way to represent the estimated damage expressed in the EMS-98 scale (Grünthal 1998). In this sense, once the vulnerability is defined by means of Eqs. (V.1) and (III.6), the mean damage grade, μ_D, D can be evaluated for different macroseismic intensities using Eqs. (III.4) and (III.8). Thus, the vulnerability curves for the historical masonry buildings of the built-up area under analysis were computed for the mean values of the vulnerability indices, ($I_{v,mean}$, V_{mean}) affected by their standard deviation value to calculate the upper and lower bounds (-2σ; $-\sigma$; $+\sigma$; $+2\sigma$), for events of different macroseismic intensity according to the EMS-98 scale (Grünthal 1998) (Fig. V.8).This kind of vulnerability curves enables to give a global appreciation of the estimated damage grade for the whole buildings stock (Fig. V.8). For instance, taking into consideration an expected earthquake of a relatively moderate intensity of VIII, the buildings which have a vulnerability index close to the mean value evaluated in each methodology, ($I_{v,mean}$, V_{mean}), it would probably suffer significant damages ($2 \leq \mu_D \leq 4$; $0.4 \leq D \leq 0.8$). However, buildings with higher vulnerability index values ($I_v > I_{v,mean}$; $V > V_{mean}$) would suffer more severe levels of damages ($4 \leq \mu_D \leq 5$; $0.8 \leq D \leq 1$). Additionally, for a macroseismic intensity of X ($I_{EMS\text{-}98}$=X) almost the entire existing historical buildings are expected to suffer a near to collapse damages in both applied methodologies (GNDT II and RISK-UE LM1).

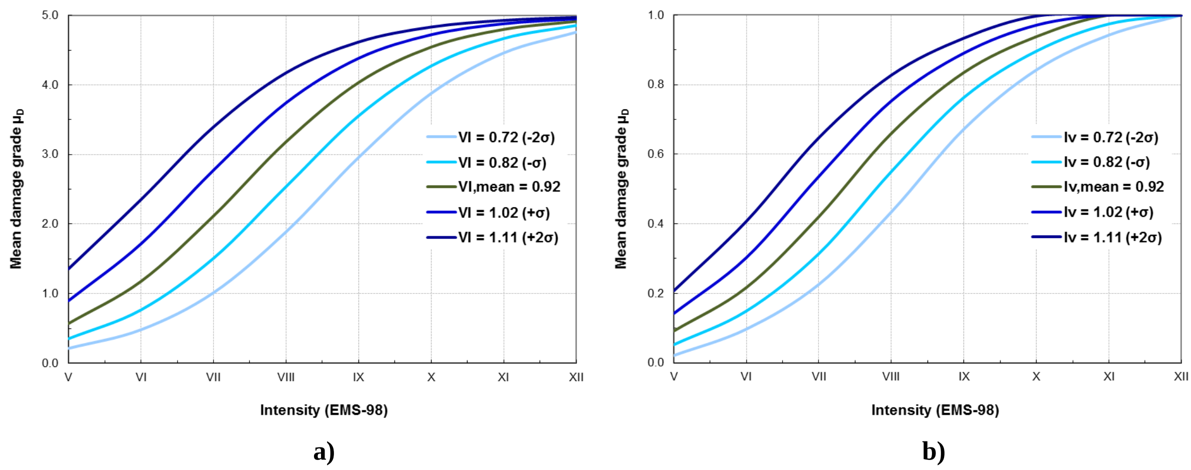


Fig. V. 8: Vulnerability curves for the entire old buildings stock of Annaba city; a) RISK-UE, b) GNDT II

Fig. V.9 presents vulnerability curves in terms of the distribution of the mean damage values μ_D and D for the studied masonry buildings. From this figure, it is clear that the two methods give almost identical results. In fact, only some slight differences are observed between the results obtained from the two approaches. For instance, for lower intensities ($I_{EMS\text{-}98}$ < VII) the modified GNDT II approach slightly underestimates the probable damage grade, while it is slightly overestimated for the higher intensities ($I_{EMS\text{-}98}$ > VII). This fact is probably due to the difference between the statistical function that each method uses to compute the mean damage: a binomial distribution for GNDT II method, and a *beta* distribution in the case of the RISK-UE ML1 method.

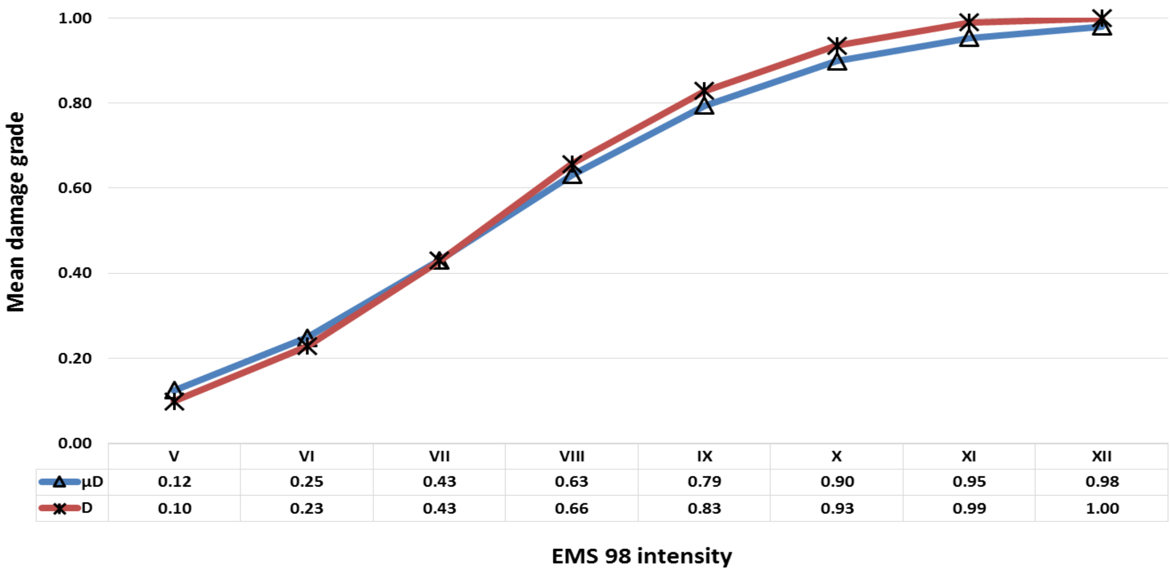
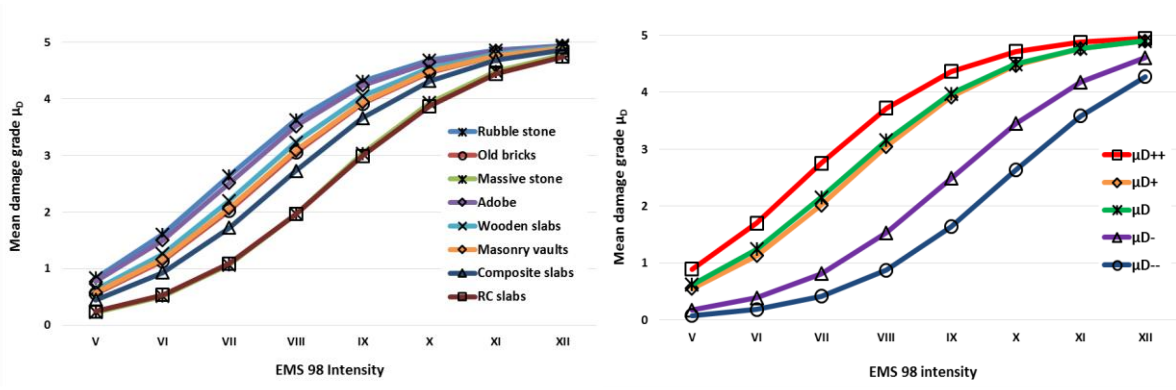


Fig. V.9: Mean vulnerability curves regarding the mean damage grades μ_D and D computed considering the RISK-UE LM1 and GNDT II methods

Fig. V.10 presents both the vulnerability curves obtained for the entire masonry building stock of the old city center of Annaba using the modified RISK-UE LM1 method with their possible range and the vulnerability curves of each buildings typology (Fig. V.10a). As can be seen in Fig. V.10b, the curve corresponding to the mean μ_D value exceeds its mean upper limit, which is explained by the high levels of vulnerability assessed. In practice, this means that significant damage can be expected, even for a moderate intensity — a fact that underlines the need for a detailed seismic risk assessment for this area.

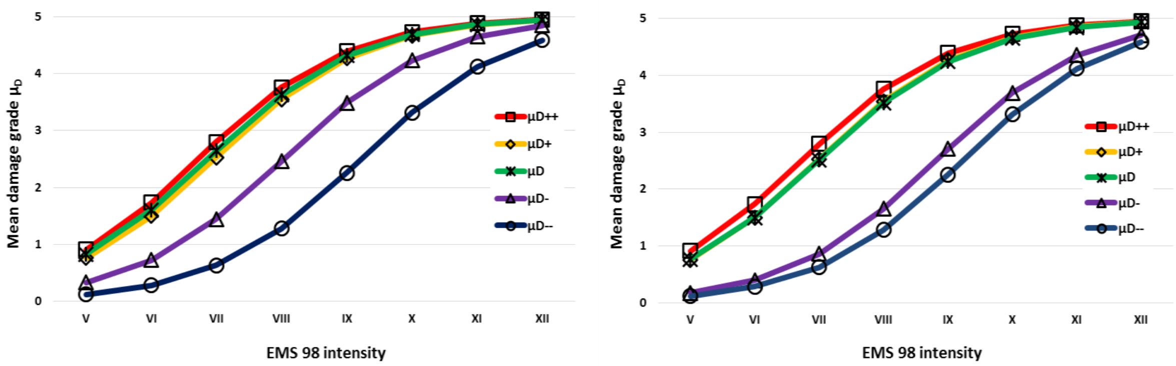


a) b)

Fig. V.10: Vulnerability curves for: a) each building typology and b) Annaba masonry building stock

It is worth mentioning that, according to the distinct vulnerability curves of each typology (Fig. V.11), the massive stone and the buildings with reinforced concrete slabs are less vulnerable. Generally, the first typology (massive stone) of buildings built with thick walls (t > 40 cm) with regular stone masonry layout are usually in moderate to good state due to the fact that they were built during the last French colonial period (end of the nineteenth century). For the second typology (URM with RC slabs), according to the EMS-98 it is classified as vulnerability class “C”, which refers to good resistance to seismic events. Moreover, some of the buildings analyzed herein have been affected by past erroneous interventions (CTC 2010), which were responsible for a significant improvement of their seismic vulnerability, such as the strengthening process where reinforced concrete columns or walls are added to strengthen the structure.

As already expected, the rubble stone and adobe buildings are the two most vulnerable typologies (Fig. V.11). This fact is particularly worrisome considering that these are prevalent in the old center of Annaba (see Fig. IV.9). Thus, it is worth presenting the vulnerability curves obtained for each one of these two typologies, with the possible upper and lower vulnerability bounds. Indeed, Fig. V.11 shows that in both the typologies the mean vulnerability curves μ_D is equal or higher than the upper possible bound, μ_D^{+}, being indeed close to the higher upper bound, μ_D^{++}. This is due to the fact that the majority of the buildings are in very poor condition; hence the structural modification scores (see Appendix) significantly increase their vulnerability.



a) **b)**

Fig. V.11: Vulnerability curves for: a) rubble stone and b) adobe buildings

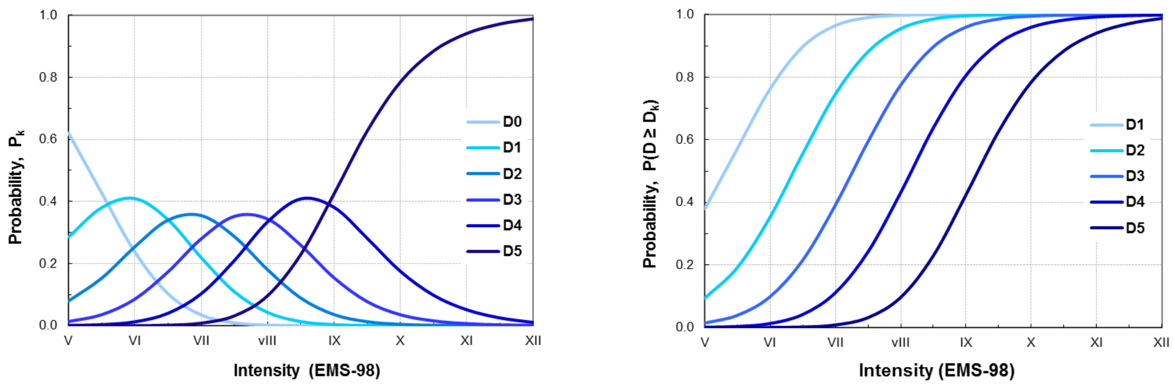
V.4 Creation of loss scenarios

Loss estimation plays an important role in the implementation of urban planning and retrofitting strategies, enabling costs to be placed alongside various beneficial measures, such as repair costs and life safety (D'Ayala *et al.* 1997). The loss estimation models are inevitably dependent on the physical damage grades, including the definition of correlations between both, the probability of a certain damage’s occurrence and the probability of different loss phenomena (Ferreira 2010). Consequently, loss assessments either in terms of collapsed or unusable buildings due to the lack of structural safety conditions, as well as the relevant death and severe injuries are in direct relation with the damage scenarios calculated hereafter based on the main proposed methodology of the modified GNDT II. In this work, the loss estimation results are organized and discussed through the construction of damage scenarios based on the global probabilistic distributions obtained for the 380 buildings evaluated. Additionally, combining the results of the probabilistic analyses with the individual building aspects and characteristics, the estimated losses are presented for each building of the built environment using a GIS tool.

V.4.1 Assessment of the damage distribution probability

To assess the damage rates for different macroseismic intensities, seismic damage needs to be quantified and measured in a standard manner. The most common approach to quantify seismic damage rates is to perform fragility analyses (Askan *et al.* 2014). Indeed, once the equivalence between both vulnerability indexes V and I_v of the selected methods RISK-UE LM1 and GNDT II respectively is performed via Eq. (III.14), the damage probabilities can be computed using a *beta* distribution function (Eq. III.12), where parameters t and r are geometric parameters associated with the damage distribution. Research carried out by Giovinazzi (2005) has shown that the *beta* distribution is the most versatile, as by controlling the shape of the distribution via the parameters t and r, it enables the fitting of both very narrow and broad damage distributions. For the definition of these parameters, the numerical damage distributions derived from the EMS-98 scale (Bernardini *et al.* 2007) are used here. The reduced variation obtained for parameter t in the numerical damage distribution justifies the adoption of a unique value of t, equal to 8. Based on this assumption and assuming that a=0 and b=6, the results of such damage probability assessment can be expressed in terms of fragility curves, which mathematically define the probability of reaching or exceeding a certain damage (Eq. III.13).

Fig. V.12 presents both probability curves for different damage grades and fragility curves obtained for the mean vulnerability index of the masonry buildings stock, $I_{v,mean}$= 57.86.

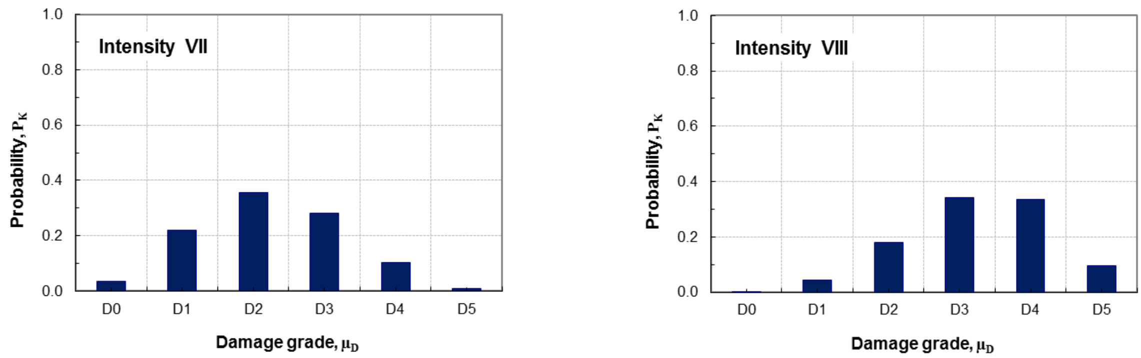


a) Probability curves

b) Fragility curves

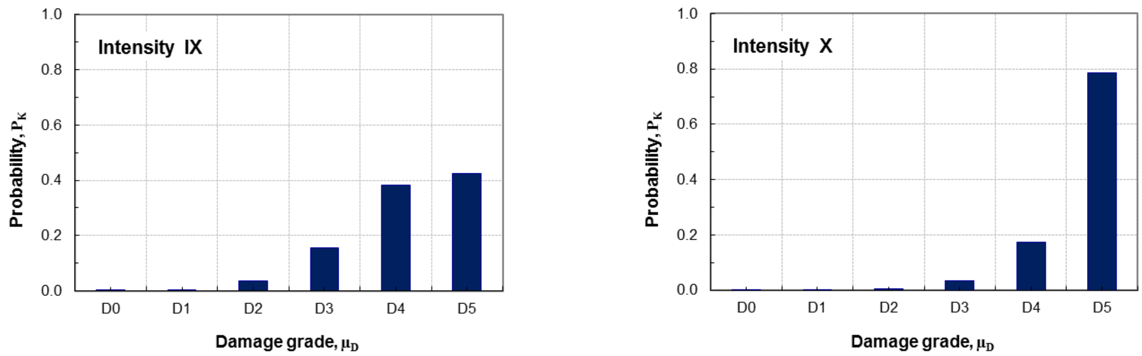
Fig. V.12: Damage of the buildings stock

Accordingly, Fig. V.13 presents the results of the damage distribution for the mean value of the vulnerability index ($I_{v,mean}$= 57.86) for the entire built-up area under study.



(a) $I_{EMS\text{-}98}$=VII

(b) $I_{EMS\text{-}98}$=VIII



(c) $I_{EMS\text{-}98}$=IX (d) $I_{EMS\text{-}98}$=X

Fig. V.13: Distribution of the damage probability for different seismic scenarios

From Fig. V.13, it is clear that the obtained results are in accordance with those concluded from the vulnerability curves of the entire buildings stock (Fig. V.12). Considering the addressed seismic scenarios, $I_{EMS\text{-}98}=VII$ to $I_{EMS\text{-}98}=X$, moderate to significant damages (D_2 to D_3) are expected for an earthquake of intensity $I_{EMS\text{-}98}=VII$ and *VIII*, and severe damages to total collapse (mainly D_4 to D_5) for events of high intensity ($I_{EMS\text{-}98}=IX$ and *X*).

As discussed by Ferreira *et al.* (2014), such kind of results can be seen as a primary tool for the developing and implementing risk mitigation and/or seismic retrofitting strategies ate the urban scale.

V.4.2 Collapsed and unusable buildings

The most frequently employed approach in the Euro-Mediterranean region are based on observed damage data after destructive earthquake events. In this respect, the loss estimation model adopted in this work was proposed by Servizio Sísmico Nazionale (SSN) based on the work of Bramerini *et al.* (1995), who approached the analysis of data associated with the probability of buildings to be deemed unusable after minor and moderate earthquakes. Although such events produce lower levels of structural and non-structural damage, higher mean damage grade values are associated with a higher probability of building collapse. The probabilities associated with the occurrence of a certain damage grade are used in the loss estimation and affected by multiplier factors, which range from 0 to 1. Eqs. (V.3) and (V.4), respectively, were used for the analysis of collapsed and unusable buildings:

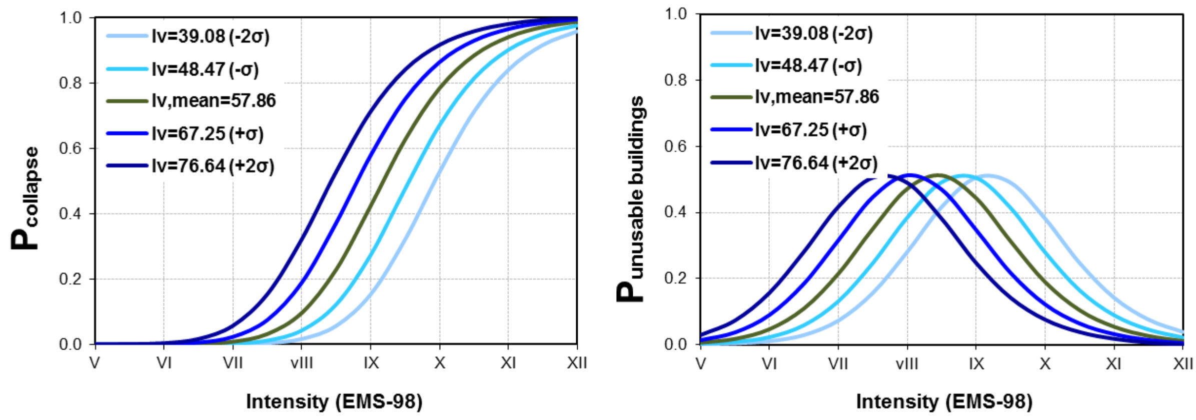
$$P_{collapse} = p_5 \tag{V.3}$$

$$P_{\text{unusable buildings}} = p_3 \times w_{\text{ub},3} + p_4 \times w_{\text{ub},4} + p_5 \times w_{\text{ub},5} \tag{V.4}$$

Where p_3 , p_4 and p_5 are the probability of occurrence for damage D_3, D_4 and D_5, whereas $w_{ub,3}$, $w_{ub,4}$ and $w_{ub,5}$ are the weight factors of this probability considered to provide the percentage of uninhabitable dwelling for each damage grade.

Following some destructive seismic events in Italy, Bramerini *et al.* (1995) have indicated that based on the surveyed data, all buildings with damage level $\mu_D \geq 4$ (D_4 and D_5) and a portion of the buildings with damage level μ_D = 3 (40%) are assumed to be unusable. Keeping this assumption, the unusable buildings rate is calculated herein, highlighting its value at each seismic intensity without taking into account the rate of collapsed buildings, which are emphatically unusable and uninhabitable. Therefore, by eliminating the probability rate of collapse from Eq. (V.4), which was already evaluated in Eq. (V.3), for the built-up area under study, the value assigned to each factor is: $w_{ub,3}$=0.4; $w_{ub,4}$=1.0; and $w_{ub,5}$=0.

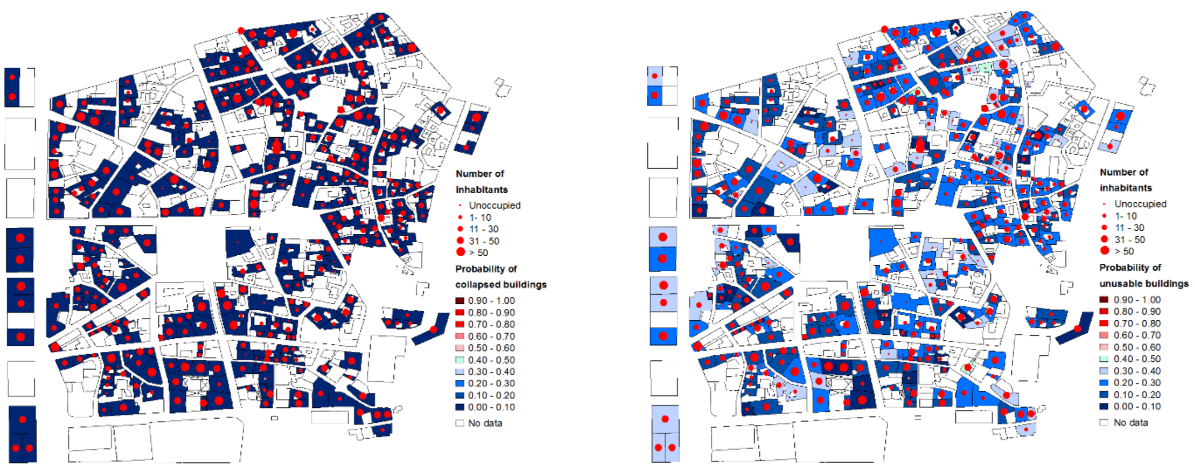
Fig. V.14 shows the results of building collapse and unusable building estimations for the mean value of the vulnerability index, $I_{v,mean}$, as well as for other representative values of vulnerability, namely: $I_{v,mean}-2\sigma_{Iv}$; $I_{v,mean}-\sigma_{Iv}$; $I_{v,mean}$; $I_{v,mean}+\sigma_{Iv}$; and $I_{v,mean}+2\sigma_{Iv}$.



(a) Probability of collapse (b) Probability of unusable buildings

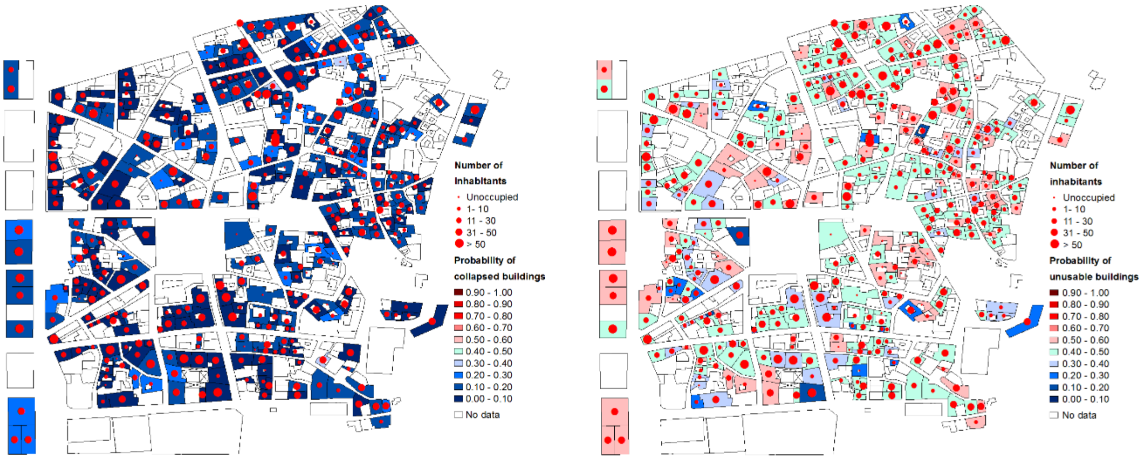
Fig. V.14: Estimative of the collapsed and unusable buildings for different seismic scenarios

From the exposed, it is worth highlighting that, as the number of buildings that suffer collapse increases with the increase of the intensity, the number of unusable buildings tends to decreases (see Table V.3). In Figs. V.15 to V.18, the probability evaluation of the collapsed and unusable buildings for seismic scenarios of intensity *VII* to *X* is presented for each building of the area under study of Annaba city.



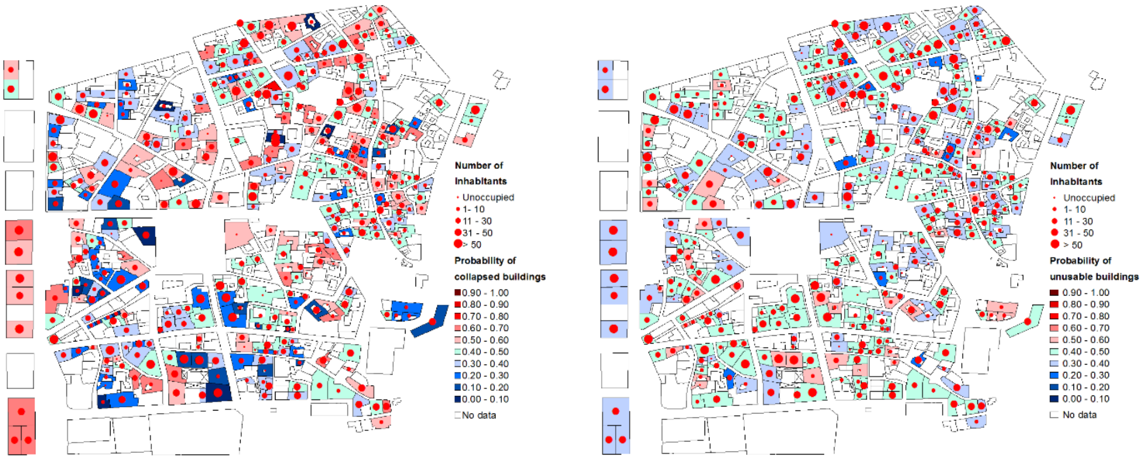
(a) Collapse (b) Unusable buildings

Fig. V.15: Evaluation of collapse probability and unusable building for a seismic scenario of $I_{EMS\text{-}98}$=VII



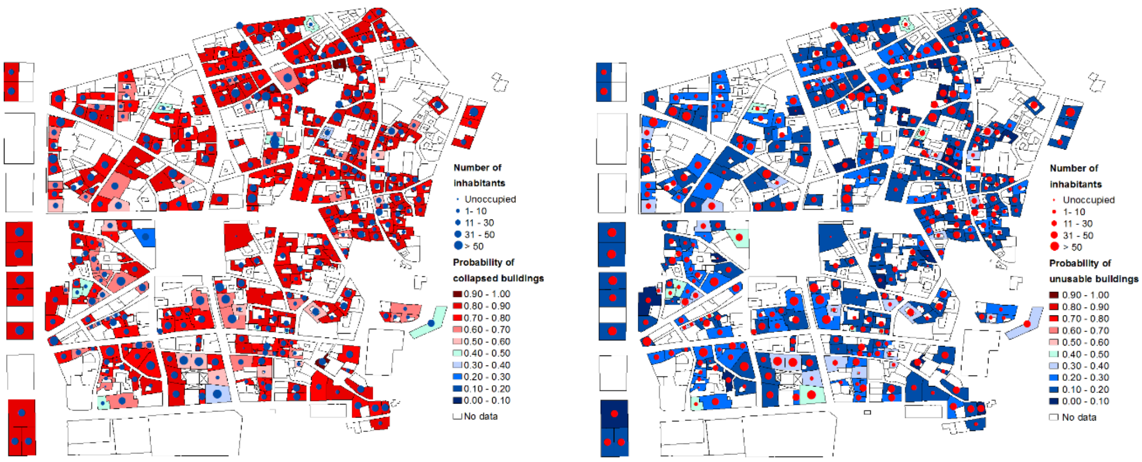
(a) Collapse (b) Unusable buildings

Fig. V.16: Evaluation of collapse probability and unusable building for a seismic scenario of $I_{EMS\text{-}98}$=VIII



(a) Collapse (b) Unusable buildings

Fig. V.17: Evaluation of collapse probability and unusable building for a seismic scenario of $I_{EMS\text{-}98}$=IX



(a) Collapse (b) Unusable buildings

Fig. V.18: Evaluation of collapse probability and unusable building for a seismic scenario of $I_{EMS\text{-}98}$=X

More representative and explicative results are listed in Table V.3 for moderate to strong intensity seismic events (*VII*, *VIII*, *IX* and *X*) (Grünthal 1998), and for the mean value ($I_{v,mean}$ =57.86) of the estimated vulnerability obtained for the 380 buildings evaluated in the old city center of Annaba.

Table V.3: Results of the collapsed and unusable buildings for different macroseismic scenarios

Total number of the assessed buildings: 380	**Intensity, *I* (EMS-98)**			
	VII	**VIII**	**IX**	**X**
Collapsed buildings	3 (0.84%)	37 (9.67%)	161 (42.45%)	298 (78.45%)
Unusable buildings	82 (21.53%)	179 (47.21%)	168 (44.31%)	72 (18.94%)
Total of affected buildings	85 (22.37%)	216 (56.88%)	329 (86.76%)	370 (97.39%)

V.4.3 Human casualties and homelessness

One of the most serious consequences of an earthquake is the loss of human life and thus one of the major goals of all risk mitigation strategies is ensuring human safety. Additionally, modeling earthquake casualty (dead and severely injured) is fundamental not only for emergency response management and for mitigation strategy planning, but also for health preparedness planning (Giovinazzi 2005).

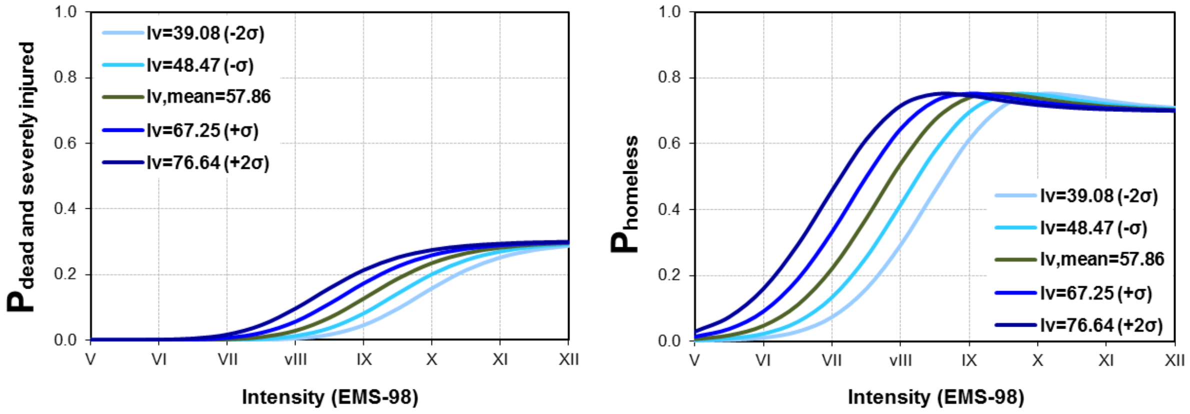
Among the various casualty rate analyses and correlation laws found in the literature, those developed by Coburn *et al.* (1992), Tiedemann (1989), HAZUS (1999) and Bramerini *et al.* (1995) are the most frequently cited. Once again, the *Servizio Sismico Nazionalle* proposal (Bramerini *et al.* 1995) was used here to guarantee the typological consistency of the loss assessment procedure. Within this approach, the estimation of the dead and severely injured rate, makes reference only to the collapsed buildings with a percentage of 30%, known that the survivors assumed to require short-term shelters. In this context, shelter need is another important parameter following an earthquake.

This study aims also at giving an estimation of the homelessness people, which is associated with uninhabitable buildings rate computed previously. The rate of unusable buildings can be therefore combined with demographic data in order to quantify number and composition of population requiring short term shelter (Giovinazzi 2005). Thus, following the same logic used for presenting and discuss the buildings loss, two rates are discussed next: probability of casualties, which is computed through Eq. (V.5), and the probability of homeless people, resorting to Eq. (V.6), both for the growing macroseismic intensity scenarios. For this purpose, the value of each multiplier factor was: $w_{ub,3}$=0.4; $w_{ub,4}$=1.0; and $w_{ub,5}$=0.7.

$$P_{\text{dead and severely injured}} = p_5 \times 0.3 \quad \text{(V.5)}$$

$$P_{\text{homeless}} = p_3 \times w_{\text{ub},3} + p_4 \times w_{\text{ub},4} + p_5 \times w_{\text{ub},5} \quad \text{(V.6)}$$

Fig. V.19 shows an estimation of the number of deaths, severe injuries and homelessness associated with the mean value of the vulnerability index, $I_{v,mean}$, and the already presented significant vulnerability values ($I_{v,mean}$-$2\sigma_{Iv}$; $I_{v,mean}$-σ_{Iv}; $I_{v,mean}$; $I_{v,mean}$+σ_{Iv}; $I_{v,mean}$+$2\sigma_{Iv}$).



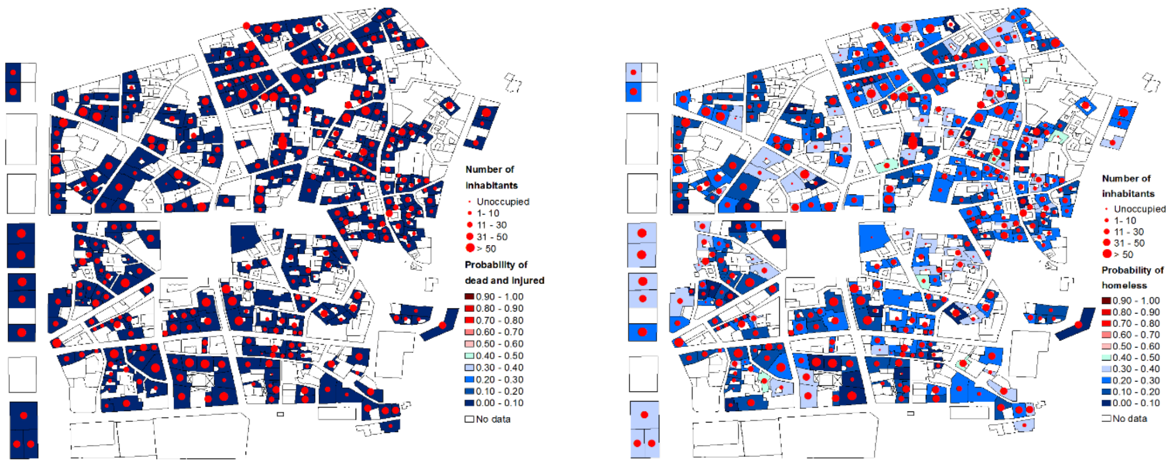
(a) Dead and severely injured

(b) Homeless

Fig. V.19: Estimation of homeless and casualty rate for different seismic scenarios

From the explained above, it is important to note that the number of homeless decreased with the intensity, as the number of dead and injured population increased.

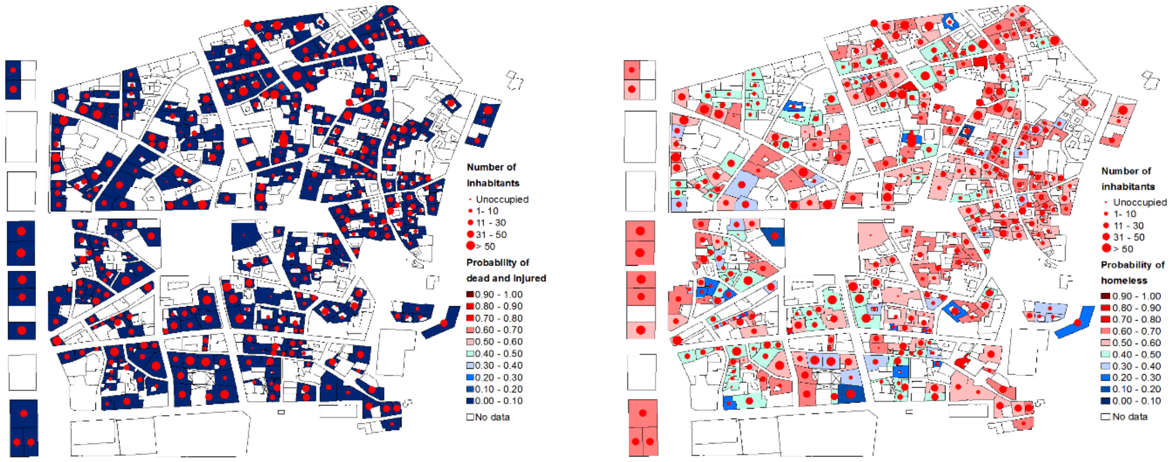
In the same of the previous loss estimations, using the developed GIS tool, an evaluation of the number of dead and injured population for seismic scenarios of intensity *VII* to *X* can be presented for each building, associating a single ID code to each polygon. Since the information is linked to all building data, it is also possible to combine different data layers, as shown in Figs. V.20 to V.23, in which the number of inhabitants is plotted.



(a) Dead and severely injured

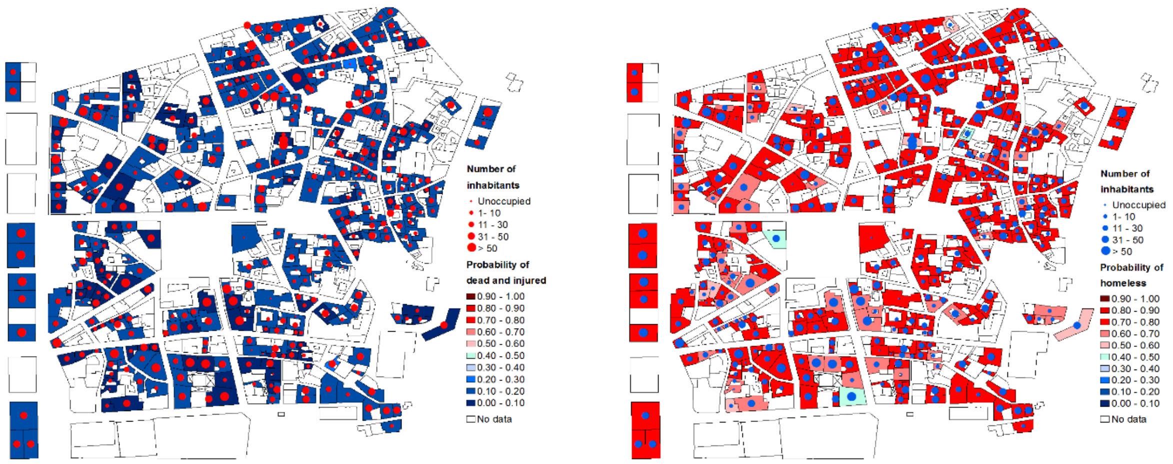
(b) Homeless

Fig. V.20: Evaluation of death rate and shelter requirements for a seismic scenario of $I_{EMS\text{-}98}$=VII



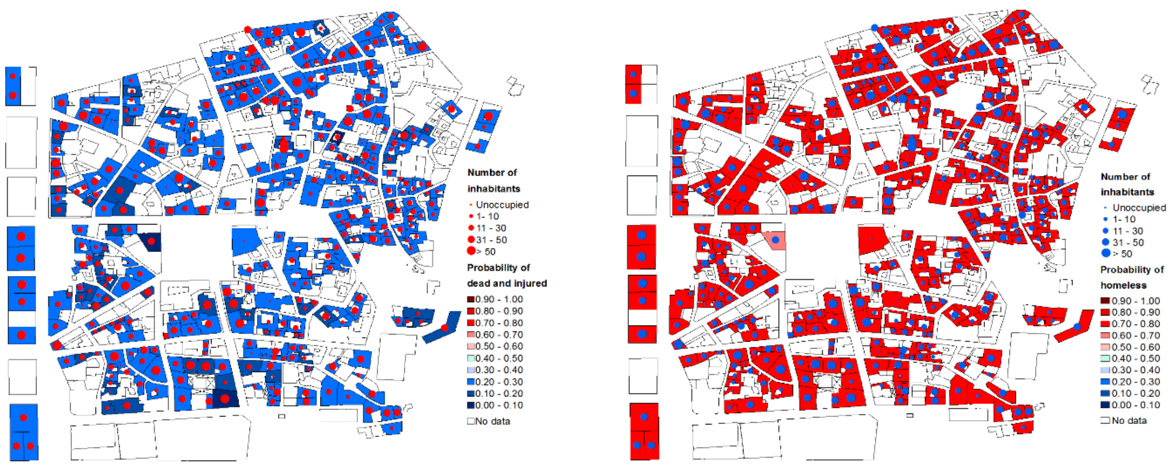
(a) Dead and severely injured (b) Homeless

Fig. V.21: Evaluation of death rate and shelter requirements for a seismic scenario of $I_{EMS\text{-}98}$=VIII



(a) Dead and severely injured (b) Homeless

Fig. V.22: Evaluation of death rate and shelter requirements for a seismic scenario of $I_{EMS\text{-}98}$=IX



(a) Dead and severely injured (b) Homeless

Fig. V.23: Evaluation of death rate and shelter requirements for a seismic scenario of $I_{EMS\text{-}98}$=X

Further results of the dead and severely injured, and homelessness inhabitants are summarized in Table V.4 for moderate to strong intensity seismic events (*VII*, *VIII*, *IX* and *X*) (Grünthal 1998), and for the mean value ($I_{v,mean} = 57.86$) of the estimated vulnerability obtained for the 380 buildings evaluated in the old city center of Annaba.

Table V.4: Results of the dead and severely injured, and homelessness inhabitants for different macroseismic scenarios

Total population of the assessed buildings: 8255	**Intensity, *I* (EMS-98)**			
	VII	**VIII**	**IX**	**X**
Dead and severely injured	21 (0.25%)	240 (2.90%)	1051 (12.73%)	1943 (23.54%)
Homeless	1826 (22.12%)	4456 (53.98%)	6111 (74.02%)	6097 (73.85%)
Total of affected inhabitants	1847 (22.37%)	4696 (56.88%)	7162 (86.75%)	8040 (97.39%)

V.4.4 Economic loss and repair cost estimation

The estimated damage grade can be interpreted to many forms, wherein the frequently used in the seismic loss studies is the so-called an economic damage index, i.e. the ratio between the repair cost and the replacement cost (building value). The correlation between damage grades and the repair and rebuilding costs are obtained by the processing of post-earthquake damage data. As shown in Table V.5, a variety of correlations have been derived in earlier studies (Vicente et al. 2010).

Table V.5: Correlation between damage levels and damage index

Damage grade, D_k	**0**	**1**	**2**	**3**	**4**	**5**
Bramerini *et al.* (1995)	0.000	0.010	0.100	0.350	0.750	1.000
ATC-13 (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 most reasonable relationship, as confirmed by the post-seismic investigation, is that which 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 (Vicente et al. 2010). The values obtained by ATC-13 (1985) and Dolce *et al*. (2006) are in agreement with these criteria. Finally, the correlation between the damage grades and the repair costs adopted in the present work, is that obtained from the analysis of the data collected using the GNDT-SSN procedure established by Dolce *et al.* (2006). The repair cost probabilities $P[R|I]$ for a certain seismic event characterized by an intensity I, can be obtained from the product of the conditional probability of the repair cost for each damage level, $P[R|D_k]$ (Dolce *et al.* 2006), with the conditional probability of the damage condition for each level of building vulnerability and seismic intensity, $P[D_k|I_v,I]$ (Vicente *et al.* 2011), given by Eq. (V.7):

$$P[R \mid I] = \sum_{D_k=1}^{5} \sum_{I_v=0}^{100} P[R \mid D_{\mathrm{k}}] \times P[D_k \mid I_{\mathrm{v}}, I] \tag{V.7}$$

Despite created in April 2006, the Unit of Work and Maintenance (UWM) of the OPRM (Office of Promotion and Real estate Management) of Annaba city was responsible by now for very few rehabilitation and strengthening processes in the historical buildings of the old city center of Annaba (about twenty operations). In fact, there is no reference to cost planned yet for these kind of operations. Consequently, to estimate the repair costs associated with the different vulnerability values used in the loss evaluation ($I_{v,mean}$-$2\sigma_{Iv}$; $I_{v,mean}$; $I_{v,mean}$+$2\sigma_{Iv}$) an average cost value per unit area of 100000 DA/m^2 (Dinar of Algeria) was considered in this work (equivalent to the 750 €/m^2 that are assumed by Ferreira *et al.* (2013) for Portugal). Fig. V.24 shows the expected global costs of repair estimated for the entire study area of 380 buildings considering different seismic scenarios.

Based on observation of Fig. V.24, it should be stressed that for intensities within the range of *V* to *IX*, the difference between the minimum and maximum repair costs estimated for the vulnerability scenarios under consideration is quite significant. This difference is much smaller for higher earthquake intensities due to the high damage levels caused by severe seismic events. Taking into account the hazard of Annaba city region, the repair cost estimations for earthquake intensities *VII*, *VIII* and *IX* are the most representative (because these are the maximum intensities felt in the region). The high repair costs for events of intensity *VII*, *VII* and *IX* confirm the rather high vulnerability of the building stock. It should also be noted that the total repair cost ratio in terms of building area and the cost of replacing the entire building stock are 23.55%, 49.69% and 78.20% for the seismic intensities of $I_{EMS\text{-}98}$ = *VII, VIII* and *IX* respectively.

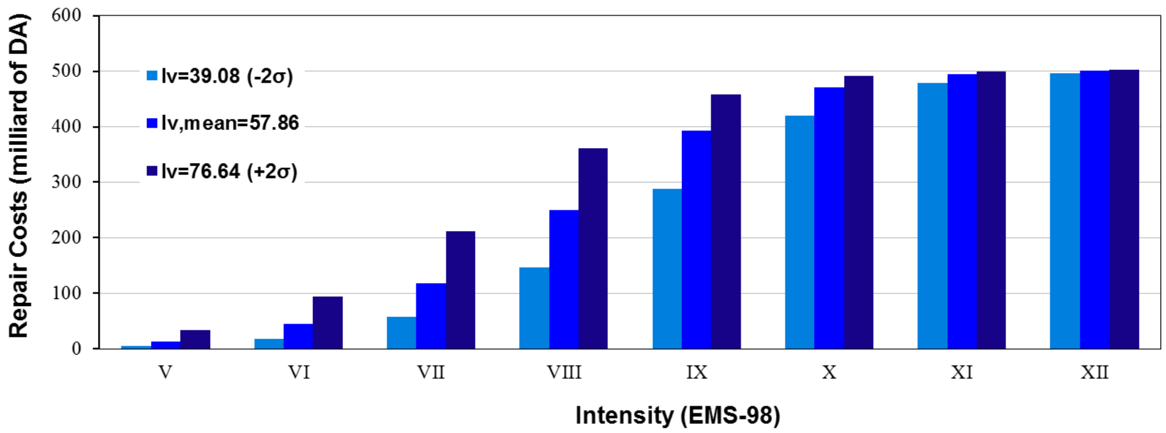


Fig. V.24: Estimation of repair costs for the studied area

Final conclusion

The analysis of the seismic risk of Annaba city was an opportunity to develop a tool for construction of scenario proper to the characteristics of these urban areas. This research presents a seismic risk study carried out taking advantage of a non-dedicated building inventory. Two different methods, modified according to the particularity of the built-up area under study, were applied. Both approaches give very representative outcomes which correlate well the features and the general fragility of the surveyed buildings. This proving the reliability of the seismic vulnerability assessment methodologies (GNDT II and RISK-UE LM1) used which are based on statistical approaches and damage observation developed essentially for the Mediterranean region are far more suitable and applicable for a large-scale analysis in Algeria, and consequently in the city of Annaba.

From a methodological point of view, the comparison of the results of the two methods has revealed two main issues:

1. The selected macroseismic methods (RISK-UE LM1 and GNDT level II) can be used as a first step on large-scale vulnerability assessments of existing buildings in seismic prone regions such as Annaba city. In fact, a very satisfactory agreement between the two methods was achieved, even with using non-dedicated data of the buildings.

2. In spite of the specificity context of the old center of Annaba city, the proposed methodology (modified GNDT II method) is suitable as a reference for the seismic risk assessment of other districts in the country, especially for the identical historic urban environment. Moreover, almost similar detailed data to the non-ad hoc CTC survey of masonry buildings should be available in the aim of getting best seismic scenarios results.

It is important to note that some of the usual interventions on historical buildings are responsible for the increase in these buildings' seismic vulnerability, namely the interruption or suppression of the resistant system on the wall bases, the increase in the number of floors and the replacement of original roof structures with heavier structures, normally out of reinforced concrete. The actual overall conservation state of the evaluated buildings is poor, presenting fragilities that could compromise their seismic behaviour, even for low to moderate intensities. Indeed, even though the city of Annaba is located in a moderate seismic hazard region, the moderate to high values of damage and loss obtained (using the two modified methods) for different earthquake scenarios revealed a considerable global seismic risk for the building stock and the historic area. Based on the scenarios analysed, the results obtained and the conclusions gathered in this study offer a great opportunity to guide the action and decision-making in the field of seismic risk in Annaba city's urban areas on two hands. On the one hand, this research could enable the development of a framework for a comprehensive database and guidance tool for local authorities responsible for rehabilitation and renewal of the historical buildings in Annaba city, in order to take into consideration the seismic strengthening measures in the interventions process. On the other hand, this work can be an important element in the context of establishment a disaster recovery and hazard mitigation strategies, prevention and emergency response for the urban areas in the short, medium, and long term. The authors believe that considering such suggestions can produce major benefits for Annaba council after eventual seismic events; the expected losses can be considerably reduced and consequently the number of expected casualties and the economic cost fall dramatically.

As recommendation, due to the uncertainties associated with statistical approaches, it is worth noting that the development of more reliable vulnerability assessment models that combine statistical and mechanical methods with respect to post-seismic data collection for the Algerian events are still an issue that must be further studied in the case of Annaba city. Moreover, including more input data such as the definition of triggers induced phenomena (i.e. taking into account the effects of site) it is certainly possible to achieve more accurate results. Additionally, the vulnerability analysis of lifelines and essential facilities, as well as a detailed analysis of the vulnerability of structures with high stakes considered fragile in the first assessment, should help providing comfort measures priority and so reduce term the exposure of people and property to seismic risk.

Application procedure

CONTENTS:

1. INTRODUCTION

The current appendix addresses the issue of processing the CTC data survey and treat each parameter included in the two selected methodologies for the vulnerability assessment and risk estimation of the historical masonry buildings located in the old town of Annaba city (Algeria).

The first part presents separately the 14 parameters, which formed the final vulnerability index of the modified “GNDT level II” method, in the aim of giving a simple use of each one according to the existing data survey. Regarding this methodology, its description and adaptation is provided herein in detail based especially on the thesis works of Vicente (2008) and Ferreira (2010).

In the same way, the second part focuses on the presentation of manner to deal with the parameters that contribute in the formulation of the final vulnerability index of the RISK-UE LM1 method, considering the existing data survey (CTC). Concerning this second method, we based our adaptation of this latter on its description defined in the handbook “WP 04: Vulnerability assessment of current buildings” (RISK-UE 2003).

2. The GNDT concept: the modified GNDT level II method

Table 1: Modified GNDT II method

PARAMETERS		Vulnerability Class C_{vi}				Proposed Weight
		A	**B**	**C**	**D**	P_i
P1	Typology of resisting system	0	5	25	45	2.50
P2	Organization of the resisting system	0	5	25	45	1.00
P3	Conventional strength	0	5	25	45	1.50
P4	Maximum distance between walls	0	5	25	45	0.25
P5	Horizontal diaphragms	0	5	25	45	1.00
P6	Number of floors	0	5	25	45	0.75
P7	Location and soil conditions	0	5	25	45	0.75
P8	Aggregate position and interaction	0	5	25	45	0.75
P9	Plan configuration	0	5	25	45	0.50
P10	Regularity in height	0	5	25	45	0.50
P11	Roof system	0	5	25	45	0.25
P12	Intervention process	0	5	25	45	0.50
P13	General stat of preservation	0	5	25	45	1.00
P14	Non-structural elements	0	5	25	45	0.25

2.1. Definition of the parameters and its application using the CTC data survey:

Parameter P1: Typology of resisting system

a) Definition

For this first parameter, we based on the famous European Macroseismic Scale EMS-98 (Grünthal 1998) which differentiates 15 separate structural typologies associated with a most likely and a range of six classes of decreasing vulnerability (A to F), as shown in the Fig. 1. As usual, for the case of our study area, we focus on the seven masonry buildings (Fig. 1) considered by this scale (EMS-98), which varied in construction materials and technology. The masonry typologies defined in the current scale are the buildings of unreinforced masonry (rubble stone and fieldstone, adobe, simple stone, massive stone, unreinforced with manufactured stone unis and unreinforced masonry with reinforced concrete floor) and reinforced or confined masonry buildings. More description of the masonry typologies are provided by Lazzali and Bedaoui (2012).

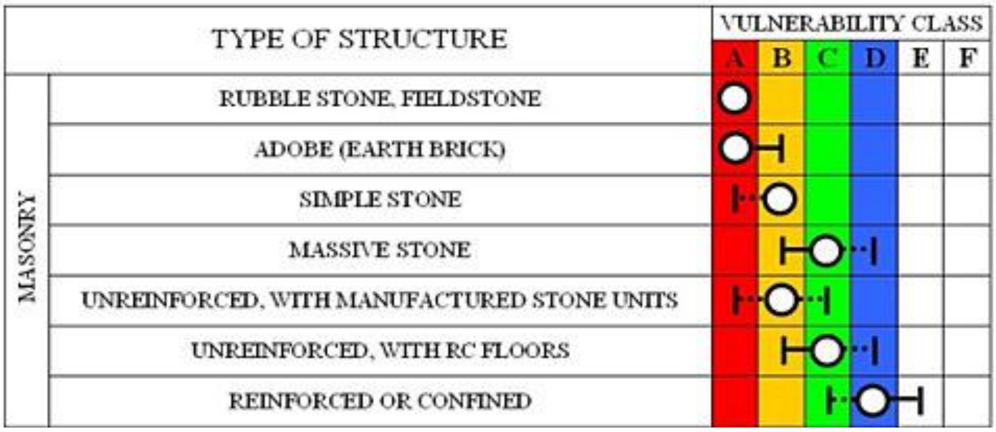


Fig. 1: Classification of Mediterranean masonry buildings Typologies and their vulnerability class

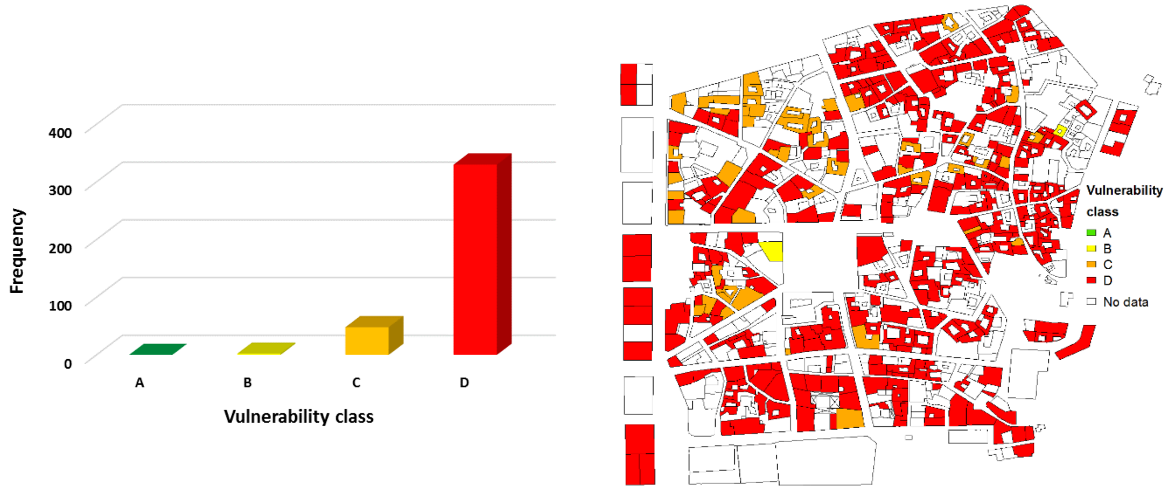
b) Application

This parameter is evaluated directly according to the EMS 98 typologies as defined above. Table 2 shows the description of this parameter for each vulnerability class considered in the GNDT approach. It is worth noting that the corresponding information of the load bearing walls typology are well inferred and available in the CTC data survey.

Table 2: Vulnerability classes definition of the parameter P1

Vulnerability class	Classification	CTC data survey
A	Typologies of the vulnerability class D	- Mixed walls (RC and a good quality of massive stone)
B	Typologies of the vulnerability class C	- Massive stone walls
C	Typologies of the vulnerability class B	- Old brick (full brick) walls
D	Typologies of the vulnerability class A	- Rubble stone walls - Adobe masonry walls

Fig. 2 presents the results of the parameter P1 (Typology of resisting system) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation described in Table 2.



a) frequnecy b) spatial distribution

Fig. 2: Typology of resisting system - P1 -

Parameter P2: Organization of the resisting system

a) Definition

This parameter evaluates the type of resistant system in terms of organization and quality of the walls of the building conception, the efficiency of connections between walls, and, if applicable, compliance with the earthquake-resistant construction codes, reinforcement and consolidation. It is essential to evaluate the distribution of shear walls, in the two principal development directions of the building, as well as the links between orthogonal walls and connecting of these to the horizontal elements, without regard to the constitution of masonry (Ferreira 2010). To assess the level of connection between orthogonal walls, particular attention should be given to the corners, with identification of the size and arrangement of stones/units. In these areas, the locking of the masonry walls are particularly important, because the careless implementation may cause the disconnection and detachment, triggered only by aging or by temperature variation (temperature cycling) (Ferreira 2010). Table 3 shows the description of the P2 parameter for each vulnerability class considered in the GNDT approach.

Table 3: Vulnerability classes definition for the parameter P2

A	This class would not be possible for the case of historical masonry buildings in fact that they should have a connection between walls in compliance with the earthquake-resistant construction codes.
B	The structure has good links with appliance and bonding between orthogonal walls, capable transmit vertical and shear loads. There are ring beams and metallic ties well distributed in sufficient numbers with good anchorage and tensioned, thus ensuring the conditions for binding and effective connection between the vertical elements and between the vertical and horizontal element.
C	The structure does not have the connections defined in class B, no or only a few, however it has good connection between orthogonal walls resistant, guaranteed by the appropriate bonding unit and all the walls
D	The Structures has no walls resistant knit. Total absence of metallic ties and ring beams.

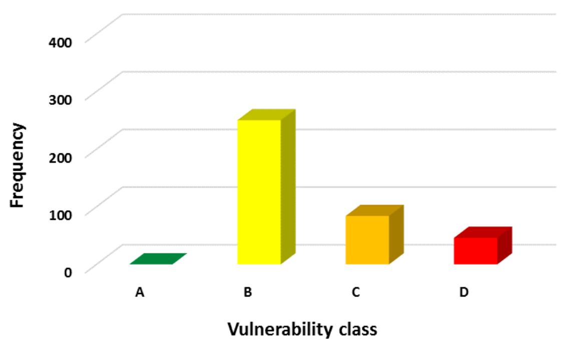
b) Application

The definition of vulnerability classes of this parameter according to the table above was not exactly informed in the CTC data survey (just some indications). In this case, we proposed a kind of adaptation based on the photos and the degradation state data (if possible, considering also the stat of the corners link and connections) of the resisting systems focusing only on the CTC survey. The four classes are presented in the Table 4 for our case study:

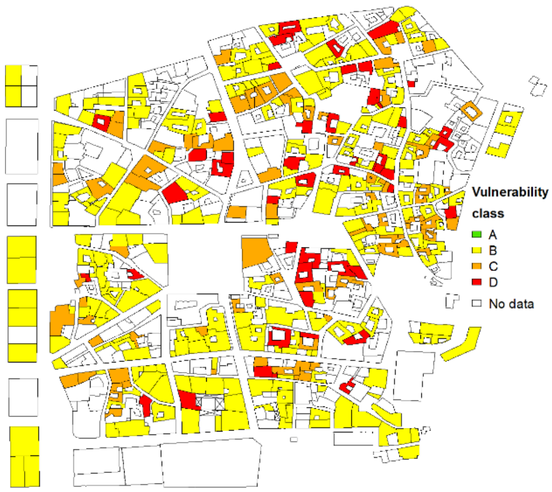
Table 4: Vulnerability classes of the parameter P2 according to CTC data survey

Vulnerability class	Classification	CTC data survey
A	Good	- Supporting elements intact - Flaking, Spalling, Chalking, Loosening
B	Slight to moderate	- Bursting, Crumbling, Rotting, corrosion, - Alteration of the joint material, Bulging wall, - Subsidence, Bulging wall - Misalignment, Cracking
C	Heavy	- Vertical default, Spillage, Disjointement, Cracking - Cracking, Dislocation, Alteration of the joint material - Localized collapse
D	Very heavy	- Localized collapse, Cracking - Dislocation, Vertical default, Spillage, Disjointement, Cracking - Localized collapse, Dislocation, Alteration of the joint material

Fig. 3 presents the results of the parameter P2 (Organization of the resisting system) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation presented in Table 4.



a) frequnecy



b) spatial distribution

Fig. 3: Organization of the resisting system - P2 -

Parameter P3: Conventional strength

a) Definition

The P3 parameter makes a significant evaluation of the overall shear strength of the building due to the seismic action based on a calculation of the resistance to lateral action. Through a fast calculation, on the assumption that the floors are infinitely rigid (still considered the absence of eccentricities in plan), quantifies the resistance to horizontal action of the structure in the two horizontal directions independently. An estimated value of resistance to lateral action of a masonry building, the weakest direction (the direction that has less resistant cross sectional area of the walls), resembling an equivalent wall (Fig. 4) (Ferreira 2010).

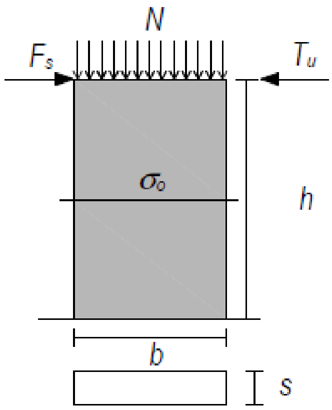


Fig. 4: In plan shear strength (Ferreira 2010)

To assess its shear resistant capacity, it is resort to **C_{conv}** coefficient, called conventional resistance that defines the resistance to shear effort in the base wall (in this case, building) and in the most unfavorable direction, taking also into account the disequilibrium between resistant areas in the two main directions line. Thus, conventional resistance **C_{conv}** is given by Eq.1 (Ferreira 2010).

$$C_{conv} = \frac{a_0 \times \tau_k}{q \times N}\sqrt{1 + \frac{q \times N}{1.5 \times a_0 \times \tau_k \times (1+\gamma)}} \tag{1}$$

$$q = \frac{(A_x + A_y) \times h \times p_m}{A_t} + p_s \tag{2}$$

$$\tau_k = 60kPa \text{ (reference value)} \tag{3}$$

$$A_{\min} = \min(A_x, A_y) \tag{4}$$

$$A_0 = A_{\min} / A_t \tag{5}$$

$$\gamma = A_{\min} / A_{\max} \tag{6}$$

$A_{\min}$: Minimum area (m^2)

$A_{\max}$: Maximum area (m^2)

A_t : Covered area (m^2)

A_x, A_y: Total area of resisting wall in the direction XX and YY, respectively $\left(m^2\right)$

h: Height between floors

p_m: Specific weight of the masonry $\left(KN/m^2\right)$

p_s: Weight per unit area of floor $\left(KN/m^2\right)$

Regarding the area of shear walls, conventionally are two orthogonal directions (Ax, Ay), considering only the vertical elements resistant walls with continuity in height in the building and which have more than 1 m of development. In the case of oblique walls, for the two main directions, the resistant area in each direction is projected in the direction of each principal axis (multiplied by $\cos^2 \alpha$, with α the value of the angle of deviation from the principal axes). In the case of buildings or in aggregate band resistant sharing walls, resistant area considered in the analysis of natural building is only half (Ferreira 2010).

It is uncommon to find when it coexists more than one type of brick in a building. In such situations, the resistance value of the characteristic section of masonry, τ_k is given as a weighted average of the resistances with the percentage of each type of existing masonry. Refers to the value of τ_k, must be carefully defined, since this parameter as well as the estimated vertical load, σ_0 (important in defining the level of installed normal stress) are the quantities that influence the calculation of conventional resistance C_{conv} (Ferreira 2010).

With respect to quantification of the amount of p_m (the specific masonry weight) and p_S (permanent floor load) technical standards documents and to support dimensioning indicate values for each constructive solution. These known quantities, p_m and p_s, and even the height between floors, h, the value of q will be defined the average weight per unit area of the whole building (sum of the weight of the floors and walls of masonry and overload regulatory). In the case of masonry buildings is very important that the indicated value for p_m be the most accurate as possible since it represents an average of a percentage of the total weight of the building (about 70%), while an estimate of p_s will not introduce such significant errors. It is noted that the value of p_s is the result of a quasi-permanent load combination in which in addition to the permanent load is considered overhead depending on the type of use of the spaces (Ferreira 2010).

Thus, the assignment of the four classes of vulnerability is defined as the quotient $\alpha = C_{conv}/C$, where C_{conv} conventional resistance is calculated using the Eq. 1. The reference value C is assumed equal to 0.4, corresponding to the calculation of maximum seismic strength to a zone of high seismicity (inevitably introducing the concept of the action) and serving just in the normalization of this parameter to define the classes. The vulnerability classes for this parameter are defined as shown in Table 5.

Table 5: Vulnerability classes definition for the parameter P3 (Ferreira 2010)

A	$\alpha \geq 1.0$
B	$0.6 \leq \alpha \leq 1.0$
C	$0.4 \leq \alpha \leq 0.6$
D	$\alpha \leq 0.4$

b) Application

Concerning this parameter, it seems so far difficult to define its vulnerability classes, in fact that almost all items included in the Eq. 1 could not be possible to deduce from the CTC data survey. Moreover, the adaptation of the resistance value of the characteristic section of masonry, $\boldsymbol{\tau_k}$ according to an existing code (like the Italian seismic code) or from other sources is an arduous task due to the specifications of the studied buildings.

In the other hand, the main parameter that could increase or decrease the overall shear strength of the building is the quality of construction material. Indeed, very significant events were recorded in the old town of Annaba city due to this fact, for instance; falling down of balconies, stairs, floors even the façade walls due to the deteriorations and alteration of the construction materials as well as the bad typologies of the resisting walls (as already mentioned, almost all the constructions typologies are the most vulnerable class according to EMS-98).

Therefore, it is worth defining the different qualities of the traditional masonry according to the same approach GNDT II in order to try inspired from their vulnerability classes, a new adaptation for our parameter P3.

The masonry found in traditional structures is very varied, with different materials components, and techniques for nesting dimensions, which give different levels of resistance. According to the work of Ferreira (2010) the quality of masonry is assessed according to three aspects:

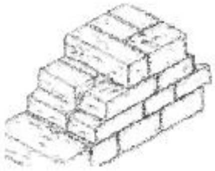
i) Homogeneity of the material constituent, shape, size and nature;
ii) Seating configuration and arrangement of the masonry;
iii) Type crosslinking in cloth wall itself.

The resistant characteristics are very dependent on the type of unit or material, and its size. The type of mortar is also an inseparable aspect, as it determining the bearing capacity of masonry, giving it a degree of monolithic. The second aspect relates to the homogeneity and regularity in the arrangement of units masonry, which is essentially of two types: a settlement with units carved with vertical joints and horizontal well defined. The third part analyzes the possible presence of cross-connecting elements such as rows, which usually joins the two pieces of wall (internal and external).

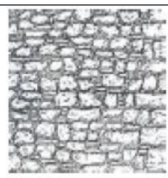
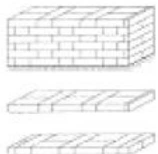
Note that the presence of horizontal rows using other materials, particularly solid brick throughout the longitudinal and transverse extension of the wall, as well as the existence of larger stones dimension along the corners and openings, situations are not considered lack of homogeneity of material or size. It is noted that the outer face of a wall of two panels, may have a more careful selection and improved apparatus of units of the internal surface. It is recommended that to view where possible, to both sides of the walls. The classification of the vulnerability classes can be made by the criteria described in Table 6 (Ferreira 2010). This classification takes account of an indirect and qualitative assessment of the degree of resistant properties of the walls and their behavior, which affect both the uniformity of load transmission, and the creation of most fragile areas of concentration of effort (creating preferential paths for transmission of load).

Table 6: Classification and description of the traditional masonry in four vulnerability classes (Ferreira 2010)

Description of the masonry Class A

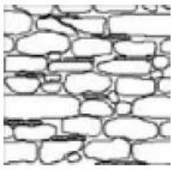
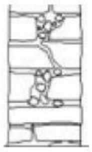
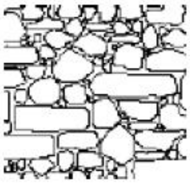
Description of the masonry	Examples of masonry Class A
Stone masonry consisting of homogeneous units (in terms of material and dimensions), and cut (parallelepiped form) with good laying and use of mortar with good quality, with vertical and horizontal joints. (A1)	 
Masonry stone of low porosity with good seating and locking with vertical joints and mortars. Mortar of good quality.	 
Masonry units with perforated clay brick or cement blocks (15 to 45% of voids) with vertical and horizontal joints and mortar of good quality.	 
Masonry of solid brick or solid blocks well established and locked with vertical and horizontal joints filled with mortar of good quality.	 

Description of the masonry Class B

Description of the masonry	Examples of masonry Class B
Stone masonry units consisting of non-homogeneous (in terms of dimensions), but well locked and arranged longitudinally and transversely. Mortar of good quality.	 
Stone masonry (just worked) with the use of stone or ceramic elements with dimensions similar to wall thickness, so that confer to the wall a cross linking throughout its thickness. Mortar of good quality.	 
Masonry based mud at a time or one and a half, of good quality with mortar.	 

Description of the masonry Class C

Description of the masonry	Examples of masonry Class C

Crudely carved stone masonry, irregularly shaped, with irregular locking and settlement. Mortar of average quality.	 
Irregular stone masonry and rounded, with cross-connection. Mortar of average quality.	 
Masonry of brick, poor settlements and mortar with poor quality.	
Irregular stone masonry without cross linking. Nesting irregularly and weak mortar quality.	 
Masonry of two panels (ornament external and internal) and composed of irregular stone fragments (stone, ceramic tiles, etc.), with a core of reasonable consistency. Irregular and settlement with average quality mortar.	
Adobe masonry based half time, with mortar of average quality.	 

Description of the masonry Class D

Description of the masonry	**Examples of masonry Class D**
Rammed earth.	
Irregular stone masonry not worked high and medium porosity. Settlement deficient (formation of voids) without elements or rows of cross linking. Mortar weak quality.	 
Clay brick masonry of poor quality, using fragments and settlement locking disabled. Mortar of poor quality.	

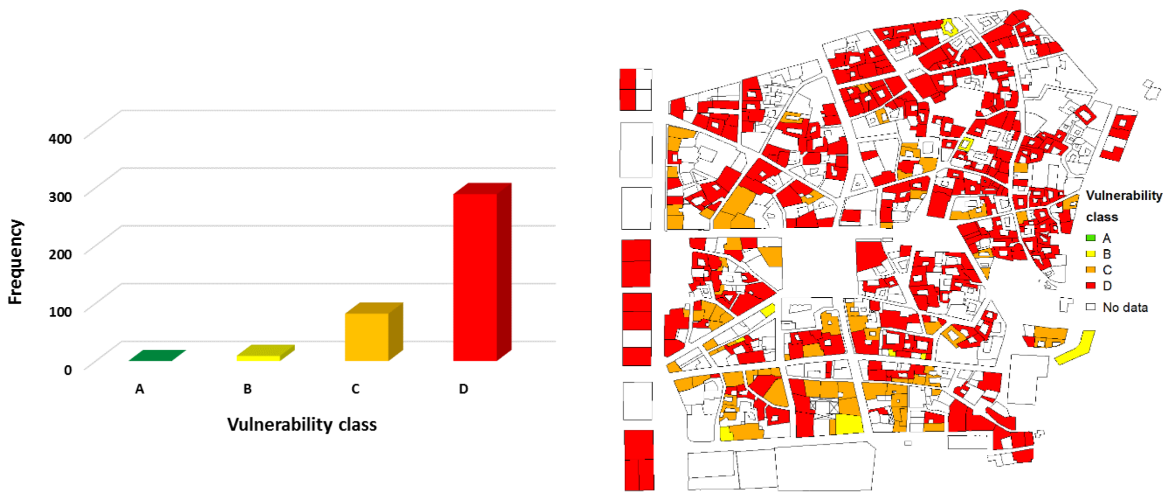
Masonry of two panels, with core half empty and unstable (no consistency). Mortar poor quality.		

Furthermore, based on the exposed above regarding the organizational quality of the structural elements inspired from the previous parameter P2, the adaptation of the current parameter P3 according to the available data survey (CTC) is presented in Table 7

Table 7: Vulnerability classes of the parameter P3 according to CTC data survey

Vulnerability class	Classification	CTC data survey
A	Good strength	-Buildings of masonry class A or B with good connection between different structural elements and thick load bearing walls (≥ 50 cm).
B	Slight strength	-Buildings of masonry class A or B with not enough link between different structural elements and their overall health could be considered slightly.
C	Moderate strength	-Buildings of masonry class B or C with bad connection between different structural elements and their overall health could be considered moderate.
D	Bad strength	-Buildings of masonry class C or D with very bad connection between different structural elements and their overall health is bad.

Fig. 5 presents the results of the parameter P3 (Conventional strength) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation illustrated in Table 7.



a) frequnecy

b) spatial distribution

Fig. 5: Conventional strength - P3 -

Parameter P4: Maximum distance between walls

a) Definition

The arrangement and distribution of resistant systems and their locking walls, particularly the peripheral walls, are important, since the level of connection between orthogonal walls and the distance between these runs the risk of triggering a mechanism to collapse out of plane of the wall (Ferreira 2010).

The criterion for this parameter P4 takes into account the distance between transverse and internal walls that stabilize the main load bearing walls. Since most buildings in historic centers are in band, this analysis is particularly important for facade walls, which generally are not well linked to longitudinal walls. This evaluation is also extended to the intermediate wall panel between floors, in which it also registers in most cases, an ineffective link. The class of vulnerability of this parameter is set to the worst situation identified for the external walls of the building envelope (see Table 8) (Ferreira 2010).

The classification is according to the geometric relationship L/s and/or h_0/s.

s : Thickness of resistant wall;

L : Maximum distance between transverse walls;

h_0 : Distance between floors or floor / roof efficiently attached to the walls.

Table 8: Vulnerability classes definition for the parameter P4 (Ferreira 2010)

Class	Description	
A	$\left(\frac{h_0}{s}\right)_{\max} \leq 10$	$\left(\frac{L}{S}\right)_{\max} \leq 15$
B	$10 < \left(\frac{h_0}{s}\right)_{\max} \leq 15$	$15 < \left(\frac{L}{S}\right)_{\max} \leq 18$
C	$15 < \left(\frac{h_0}{s}\right)_{\max} \leq 20$	$18 < \left(\frac{L}{S}\right)_{\max} \leq 25$
D	$\left(\frac{h_0}{s}\right)_{\max} > 20$	$\left(\frac{L}{S}\right)_{\max} > 25$

In case of reductions in thickness with reasonable extension in the walls, i.e., more than 30% of the thickness in length and/or greater than 1/3 of its height dimensions recesses forming the local fragility, that worsens the vulnerability class assigned according to the above criteria in Table 8 (e.g. A $\rightarrow$ B).

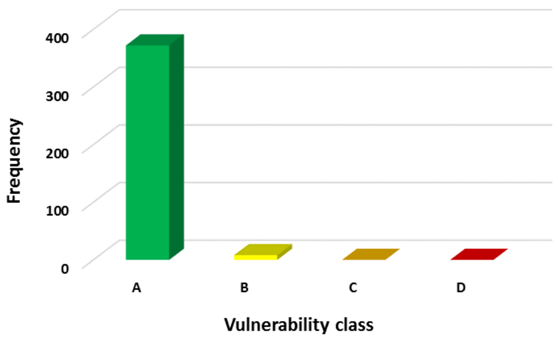
b) Application

Due to the lack of data about the maximum distance between walls in the existing CTC data and the accurate information regarding the distance between floors led us to limit our adaptation only in the first part of the classification (Table 9). However, for the thickness of the bearing walls which is also do not exist in the current database, an in-situ inspection of some buildings and a checking of their architectural plans lead to propose a fix thickness value of 40 cm for all the buildings under study.

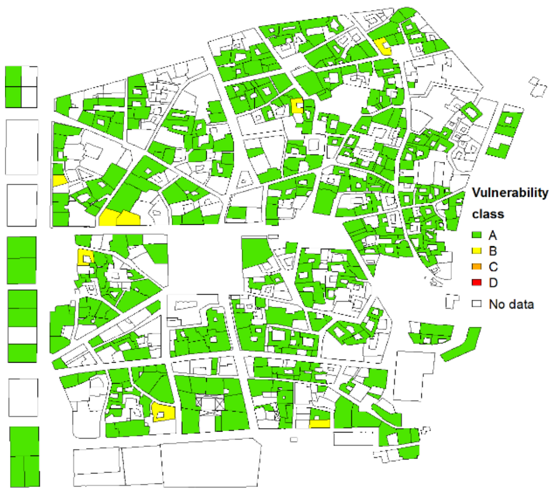
Table 9: Vulnerability classes of the parameter P4 according to CTC data survey

Vulnerability class	CTC data survey
A	$\left(\frac{h_0}{s}\right)_{max} \leq 10$
B	$10 < \left(\frac{h_0}{s}\right)_{max} \leq 15$
C	$15 < \left(\frac{h_0}{s}\right)_{max} \leq 20$
D	$\left(\frac{h_0}{s}\right)_{max} > 20$

Fig. 6 presents the results of the parameter P4 (Maximum distance between walls) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation exposed in Table 9.



a) frequnecy



b) spatial distribution

Fig. 6: Maximum distance between walls - P4 -

Parameter P5: Horizontal diaphragms

a) Definition

The quality and type of structural system of the floors has a remarkable influence on the global behavior of buildings. It is very important that they are well connected to the walls to give them the vertical and horizontal loads and these, in its turn, to the foundations. When the connection between the horizontal elements and the walls do not work effectively, movements induced on the walls and can trigger the shutdown defilade the bars / joist hangers of the floors and walls collapse thereof. A poor connection between the floors and walls prevents the continuous distribution of shear stress by shear walls, creating distortions and deformations in the walls higher compared with that found in cases of rigid floors. The deficiency of these connections creates instability in the structure, floors losing its ability to lock walls (increasing its slenderness and consequently decreasing their capacity).

The floors with insufficient stiffness in its plan inducing a fragile structure behavior, not mobilizing the response of the walls fairly (Ferreira 2010).

The classes of vulnerability for this parameter are set as shown in Table 10 (Ferreira 2010). In addition, Vincent (2008) proposing aggravate grade rating of this parameter, depending on the condition of the floor, as this affects their connection conditions to the walls (deterioration by biological action or rotting) and the stiffness of the slab itself. In the same work is further described the establishment of some common types of flooring, classifying them according to their deformability.

There are also two exceptions to this parameter in its classification (Ferreira 2010):

- The floors which have height differences, usually inducing a high concentration of efforts in the resistant walls, especially for horizontal actions, proposing this methodology in the presence of gaps, will further worsens the classification obtained from Table 10 of a class of vulnerability (except the obvious exception has already been classified as D);

- For the case of buildings with floors in reinforced concrete or other similar heavy and rigid solution, wherein the resilient structure of masonry walls was rated according to the parameter P2 of class C or D (types of low strength and stiffness compared to horizontal structures), in these cases, the class of vulnerability will to attribute to D.

Table 10: Vulnerability classes definition for the parameter P5 (Ferreira 2010)

Flooring	Class	FSA or DA or LS
Rigid or semi-rigid and well connected	A	B
Deformable and well connected	B	C
Rigid or semi-rigid and badly connected	C	D
Deformable and badly connected	D	-
FSA: fragility of floor in supporting area; **DA:** signs of deformation, rotting, shrinkage or severe distortion; **LS:** lack of traffic safety		

b) Application

For this parameter, almost we took the same definition of the previous table including some modifications or adaptations according to CTC data survey due to the inaccuracy of information of the connection between the different elements of the structure. The classification is presented in the Table 11.

Table 11: Vulnerability classes of the parameter P5 according to CTC data survey

Vulnerability class	Classification	CTC data survey	With D.S

A	Rigid	-Reinforce concrete slabs	B
B	Semi-rigid	-Composite steel (beams) and masonry (vaults or not) slabs -Masonry vaults slabs	C
C	Semi-deformable	-Wooden slabs connected to the bearing walls with steel tie beams	D
D	Deformable	-Slab with structure and wooden secondary elements	-

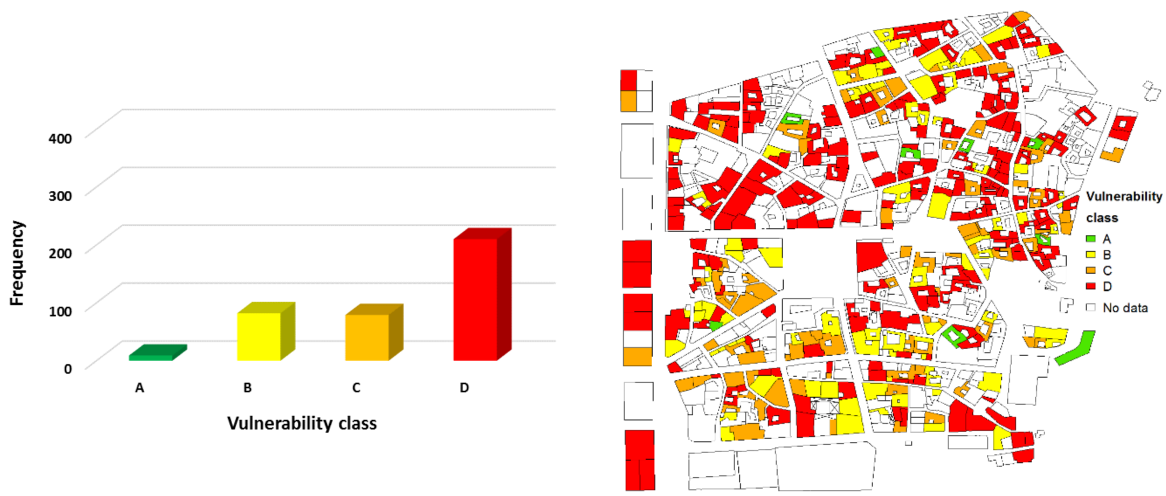
D.S: we considered only the case of heavy or very heavy degradation state to increase the vulnerability class.

The degradation state of the horizontal system (diaphragm) is defined in the same way of the resisting system, thus according to the degradation information cited in the CTC diagnosis about this element, we suggested the following classes shown in Table 12:

Table 12: Classes of degradation state of the horizontal diaphragm system

Vulnerability class	Classification	CTC data survey
A	Slight	- No degradation - Bulging, Corrosion, Rotting
B	Moderate	- Subsidence, Rotting, Undulation - Bulging, Subsidence
C	Heavy	- Bending, Buckling, - Breaking
D	Very heavy	- Breaking, Warping - Warping, Bending, Buckling, Subsidence

Fig. 7 presents the results of the parameter P5 (Horizontal diaphragms) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation based on the complementary Tables 11 and 12.



a) frequnecy b) spatial distribution

Fig. 7: Horizontal diaphragms - P5 -

Parameter P6: Number of floors

a) Definition

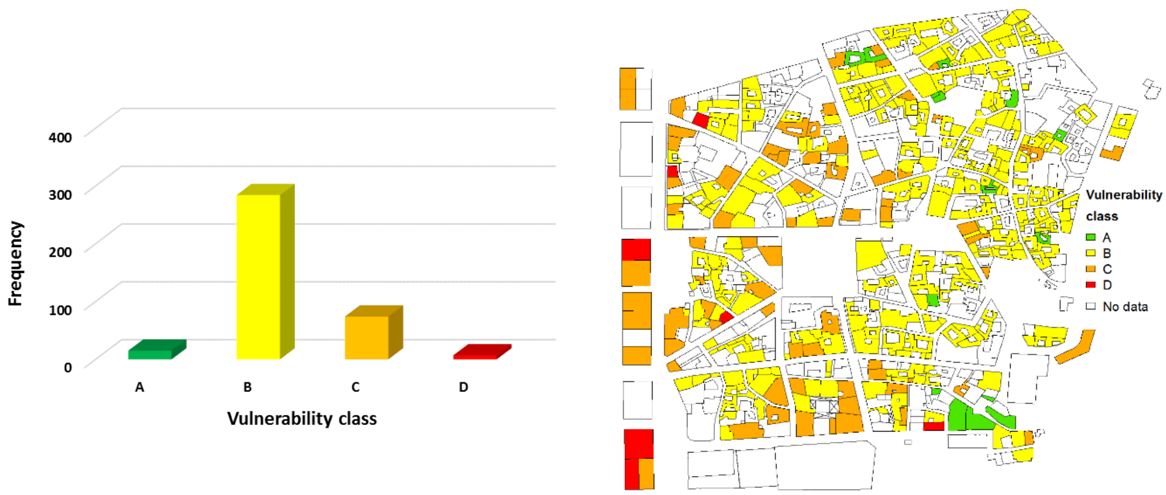
This P6 simple assignment, parameter associates greater vulnerability to buildings of greater height. This parameter is not intended to evaluate the irregularity in height, or associating the estimate of the frequency or stiffness with height, but rather expose the concept of relativity. The highest masonry buildings tend to be more vulnerable and susceptible than low buildings. The proposed classification for this parameter is shown in Table 13 (Ferreira 2010).

Table 13: Vulnerability classes definition for the parameter P6 (Ferreira 2010)

Vulnerability Class	Classification
A	Building with 1 floor
B	Building with 2 or 3 floors
C	Building with 4 or 5 floors
D	Building over 5 floors

b) Application

The data about the number of floors is well and directly informed in the first part of the diagnosis of the CTC survey, therefore the same vulnerability classes' definition is took for this parameter (Table 13). Fig. 8 presents the results of the parameter P5 (Number of floors) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the CTC data survey based on Table 13.



a) frequnecy b) spatial distribution

Fig. 8: Number of floors - P6 -

Parameter P7: Location and soil conditions

a) Definition

This parameter evaluates the importance of factors such as: topography of the building envelope (pending Land, *p*), type and consistency of ground foundation, existence of foundations and height difference between these (Δh), slope gradient and possible presence pulse unbalanced lands (Fig. 9). In this simplified procedure, given the difficulty of assessing the interaction ground-building in each case, were taken over by Vicente (2008) some simplifications in depth inspection, adaptable to the operational needs of the methodology, some procedures may be used. The vulnerability classes are defined in Table 14 (Ferreira 2010).

Fig. 9: Location of the structure in various slope of the land (Ferreira 2010)

Table 14: Vulnerability classes definition for the parameter P7 (Ferreira 2010)

Description	Classification		Class
Rock with or without foundation	$p \leq 10$	-	A
	$10 < p \leq 30$	-	B
	$30 < p \leq 50$	-	C
	$p > 50$	-	D
Released without impulse, with foundation rock	$p \leq 10$	$\Delta h = 0$	A
	$p \leq 10$	$0 < \Delta h \leq 1$	B
	$10 < p \leq 30$	$\Delta h \leq 1$	B
	$30 < p \leq 50$	$\Delta h \leq 1$	C
	$p > 50$	-	D
	-	$\Delta h > 1$	D
Released without impulse and without foundation	$p \leq 10$	$\Delta h = 0$	A
	$p \leq 10$	$0 < \Delta h \leq 1$	B
	$10 < p \leq 20$	$\Delta h \leq 1$	B
	$20 < p \leq 30$	$\Delta h \leq 1$	C
	$p > 30$	-	D
	-	$\Delta h > 1$	D
Rock, with or without foundation	$p \leq 50$	$\Delta h \leq 1$	C
	$p > 50$	-	D
	-	$\Delta h > 1$	
Released with impulse without foundation	$p \leq 30$	$\Delta h \leq 1$	C
	$p > 30$	-	D
	-	$\Delta h > 1$	

b) Application

This parameter is evaluated according to the information of the slope degree informed in the CTC data survey. However, the designation used for the type of soil is proposed according to the Algerian seismic code (S1, S2, S3, S4) (CGS 2003). Table 15 shows the definition of the vulnerability classes for the parameter P7.

Table 15: Vulnerability classes of the parameter P7 according to CTC data survey

Description	Classification	Vulnerability class
Soil type S1 with or without foundation	$p \leq 10$	A
	$10 < p \leq 30$	B
	$30 < p \leq 50$	C
	$p > 50$	D
Soil type S2 or S3 with foundation	$p \leq 10$	A
	$10 < p \leq 30$	B
	$30 < p \leq 50$	C
	$p > 50$	D
Soil type S2 or S3 without foundation	$p \leq 10$	A
	$10 < p \leq 20$	B
	$20 < p \leq 30$	C
	$p > 30$	D
Soil type S4 with foundation	$p \leq 50$	C
	$p > 50$	D
Soil type S4 without foundation	$p \leq 30$	C
	$p > 30$	D

Fig. 10 presents the results of the parameter P7 (Location and soil conditions) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation of Table 15.

a) frequnecy b) spatial distribution

Fig. 10: Location and soil conditions - P7 -

Parameter P8: Aggregate position and interaction

a) Definition

The assessment of the structural regularity of the building inserted into block (adjacent to buildings or other resistant elements which shares with the neighboring buildings), should not be analyzed individually. One should take into account the interaction with the structural unit to which it belongs (group of buildings) with respect to its seismic response, that is, the demands of deformation due to the interaction point (Ferreira 2010).

The response of the building to the horizontal action is influenced by its insertion into an aggregate of buildings, allowing the confinement and interaction produced beneficial or harmful act in certain situations, such as the analyzed building located on street corner, confined on both sides or only (see Table 16) (Ferreira 2010).

The presence of floors in solid or lightened concrete (usually pre-stressed profiles vaults ceramics), or mixed (steel-concrete) in buildings adjoining masonry buildings with floors of wood, lead to an effect known as pounding. Equal to or greater unevenness 0.5m are considered sufficient to cause this phenomenon, translated in this parameter for the aggravation of the class of vulnerability considered (Table 16) (Ferreira 2010).

Table 16: Vulnerability classes definition for the parameter P8 (Ferreira 2010)

Localization	Class	Difference level of the floor
Building in the middle	A	B
Isolated building	B	-
Building in the corner	C	D
Building in the extremity	D	-

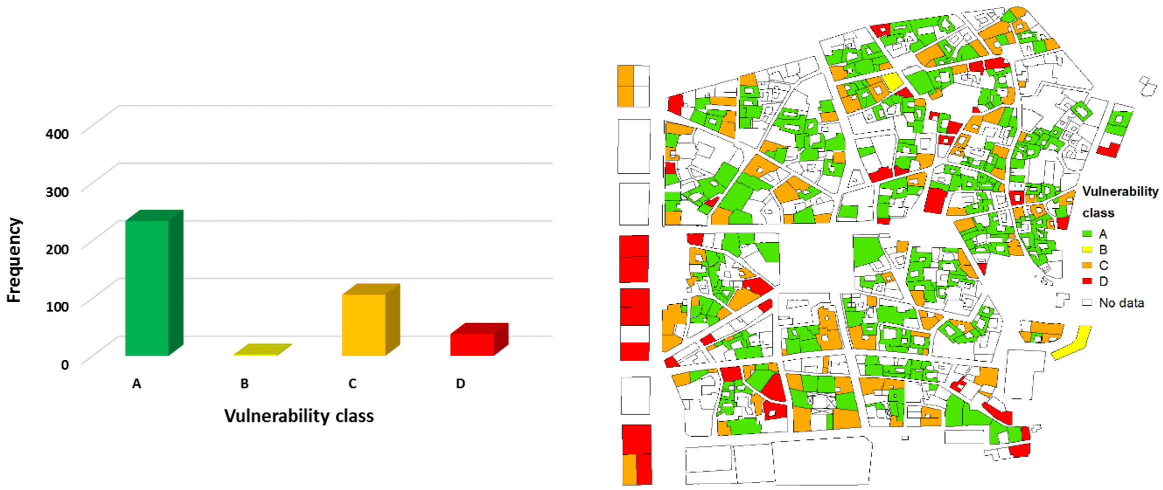
b) Application

The aggregate position of the building is accurately defined in the CTC data survey in the description part. Furthermore, this parameter is easy to check from other sources like google earth, or land use plans of the area under study (DUC 2006). Table 17 shows the definition of the vulnerability classes for the parameter P8.

Table 17: Vulnerability classes of the parameter P8 according to the CTC data survey

Vulnerability class	CTC data survey
A	Limited by three buildings or by two buildings in parallel sides
B	Isolated building
C	Limited by two buildings in the tow opposite sides
D	Limited by one building in one side

Fig. 11 presents the results of the parameter P8 (Aggregate position and interaction) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation presented in Table 17.



a) frequnecy b) spatial distribution

Fig. 11: Aggregate position and interaction - P8 -

Parameter P9: Plan configuration

a) Definition

The shape and disposition in plan of the resistant building system are aspects that affect the structural performance and consequently the seismic vulnerability. For this parameter were proposed two levels of evaluation, allowing two levels of approach in the assignment of class, as explained below (Vicente 2008).

The irregularity in plan can be evaluated expeditiously; using geometric relationships based on criteria of symmetry in plan dimensions of the building envelope (see Fig. 12) (Vicente 2008). Regarding the use of the geometric criteria (defined by β_1 and β_2 indicators), it is noted that for β_2 in the case of an inserted assessed whether into an aggregate building, the adjoining buildings will give this geometric irregularity by partial enclosure of the building in question.

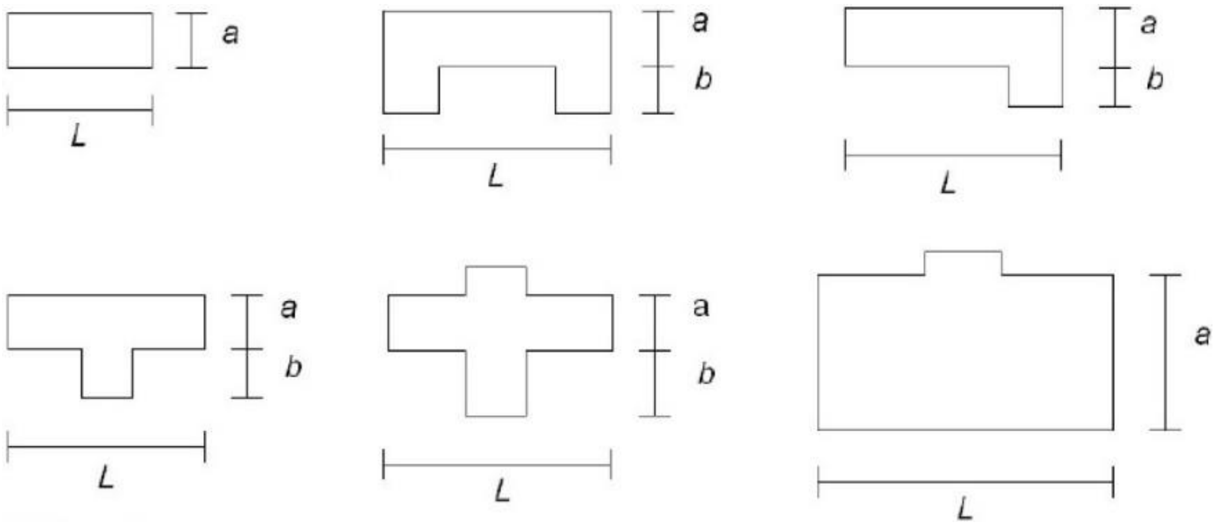


Fig. 12: Some common geometries of buildings in plan (Vicente 2008)

A more rigorous approach to the assessment of irregularity in plan is to estimate the eccentricity between the center of mass and center of stiffness (see Fig. 13) (Vicente 2008).

This evaluation process more costly, can be applied when there is geometric information about the building (architectural survey). To define the classes of vulnerability to the more detailed approach, the author has established limits for the eccentricities (distance between the center of stiffness and the center of mass) (Ferreira 2010).

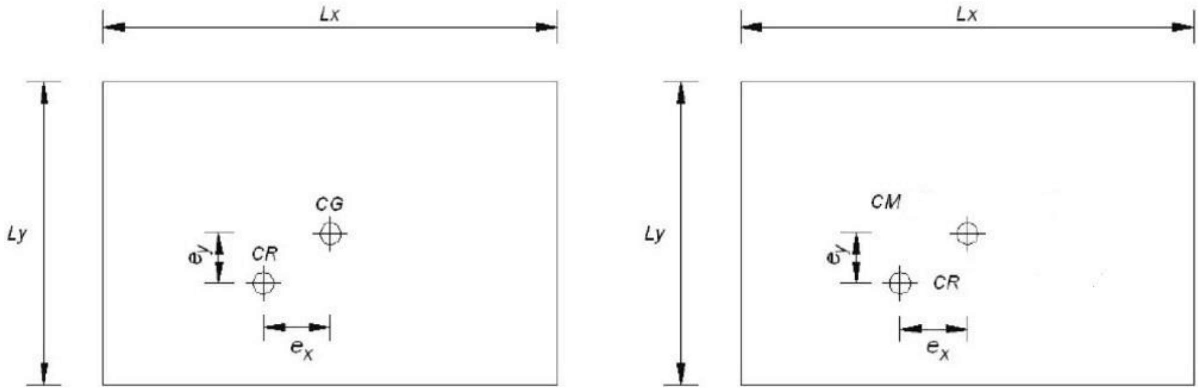


Fig. 13: Eccentricity in the two horizontal plan directions (Vicente 2008)

The selection of the class was based on the verification of the worst conditions for the two levels of detail established by two criteria (see Table 18) (Ferreira 2010).

Table 18: Vulnerability classes definition for the parameter P9 (Ferreira 2010)

Class	Application		Criteria of the eccentricity
A	$\beta_1 \geq 0.75$	$\beta_2 \leq 0.1$	Less than 10% of the largest dimension in plan
B	$0.5 \leq \beta_1 < 0.75$	$0.1 < \beta_2 \leq 0.2$	Between 10 and 20% of the largest dimension in plan
C	$0.25 \leq \beta_1 < 0.5$	$0.2 < \beta_2 \leq 0.3$	Between 20 and 30% of the largest dimension in plan
D	$\beta_1 < 0.25$	$\beta_2 > 0.3$	More than 30% of the largest dimension in plan

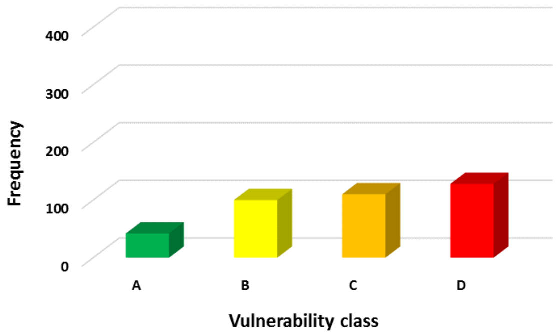
Known that according to the figure:

$$\beta_1 = \frac{a}{L} \qquad \text{and} \qquad \beta_2 = \frac{b}{L} \tag{7}$$

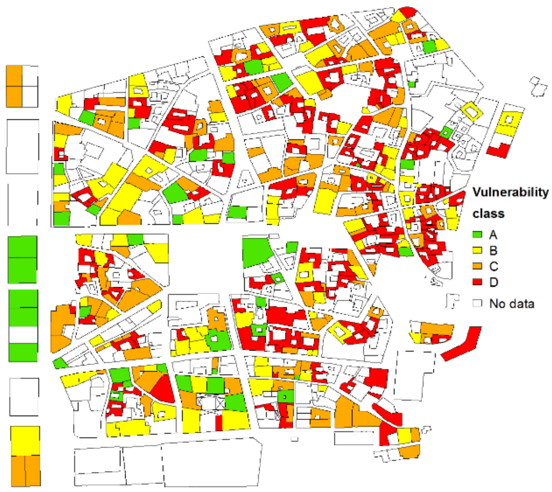
This parameter penalizes buildings with non-symmetric geometry in plan, elongated shapes with great development in one direction compared to another, and, overhanging bodies that can cause overall torsional deformation and greater demands on resistant elements (Ferreira 2010).

b) Application

Based on the dimensions given in the sketch figures in the CTC data, we could evaluate the degree of irregularity of the buildings according to the classification presented in the Table 18 based only on β_1 (due to the difficulty in application of β_2). In some cases, the sketch of the CTC data do not include the dimensions value, in this case we completed them based on the land use plans which were designed with the real dimensions of the buildings (DUC 2006). Fig. 14 presents the results of this parameter P9 (Plan configuration) in terms of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the CTC data survey.



a) frequnecy



b) spatial distribution

Fig. 14: Plan configuration - P9 -

Parameter P10: Regularity in high

a) Definition

This parameter evaluates the area variation between two consecutive floors. Simplistically, the assessment of irregularity in height can be done by estimating the change in area between floors, $\pm \Delta A / A$ (%), where A is the floor area. The choice of the class follows the criteria presented in Table 19. The classification of this parameter corresponds to the most unfavorable condition (Ferreira 2010).

There are also a few exceptions: in the case of bearing walls of the building which are made from various materials used in terms of different levels, and that this significantly alter the rigidity and strength to the walls, it should penalize the previously assigned class using the simplified criteria in Table 19, according to the following criteria (Ferreira 2010):

1. If due to the simplified criteria the building is classified as Class A or B should be considered as belonging to the class C;

2. If due to the simplified criteria the building is classified as Class C, shall be considered as belonging to the class D.

The addition of floors after the original construction typically is discontinuous in terms of material and consequently stiffness. This situation is particularly aggravated by poor connection conditions to the original structure, increasing their vulnerability. For buildings under these conditions is proposed to be classified as Class D (Ferreira 2010).

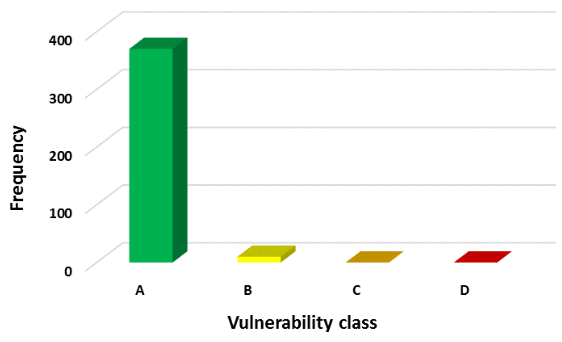
In the case of buildings where the ground floor or other high level (less often the case) have been suppressed or interrupted shear walls or have the opening of large voids (most frequent situation at the ground floor level) was carried out, introducing an important variation of stiffness is attributed to class D (Ferreira 2010).

Table 19: Vulnerability classes definition for the parameter P10 (Ferreira 2010)

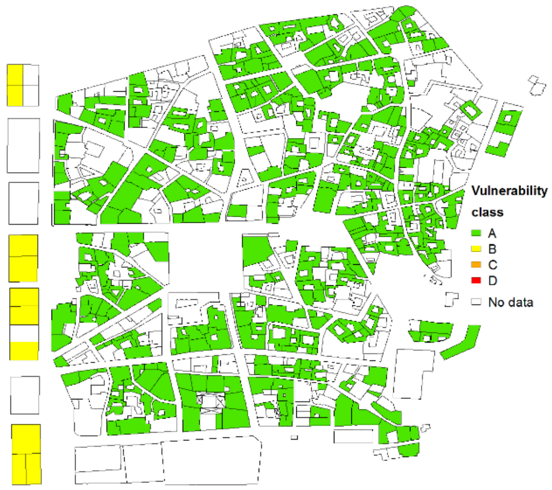
Class	Description
A	Building with mass distribution and constant floor area in all its height. Building, with a reduction in area of less than 10% plan
B	Building with a variation of area greater than 10% and less than 20% of the area in plan. Building with a tower height of less than 10% of the total height of the building. Building with gallery or small arch (corresponding to less than 10% of the total area of the floor)
C	Building with a variation of area greater than 20% of the plan area. Building with a tower height of more than 10% and less than 40% of the total height of the building. Building with gallery or arcade area with more than 10% plan and less than or equal to 20% of the total floor area
D	Buildings with setbacks representing a variation of area exceeding 30% of the area plan. Building with a tower height of less than 40% of the total height of the building. Buildings with gallery or arcade with an area greater than 20% of the total floor area

b) Application

Although, the parameter of the regularity in high generally is not filled in the CTC data survey, however based on the same vulnerability classes defined in Table 19 and on a rapid in-situ checking, we concluded that almost all the buildings have a good configuration in elevation, except some of them where a moderate irregularity in elevation is shown (class “B” according to the Table 19) due to the presence of loggias in their first floor. Usually this kind of constructions were built by the French colonial. Fig. 15 presents the results of the parameter P10 (Regularity in high) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the in-situ checking.



a) frequnecy



b) spatial distribution

Fig. 15: Regularity in high - P10 -

Parameter P11: Roof system

a) Definition

The criteria used in the definitions of this parameter are primarily related to the structural configuration of the roof (weight, dimension and will support conditions on the perimeter). The possibility of the horizontal roof causing impulses on the walls is certainly a constraint on the performance aspect of the building, which greatly depends on the structural solution of the roof, the existence of binding elements of the roof to the wall, the possible presence of a perimeter strap or tie rods and further, of its condition (Ferreira 2010).

The impulsive nature of the roof is especially important for seismic actions because they may increase the impulses on the facade walls, eventually causing the collapse out of plan. In addition, knowing the type of coverage and identify the existence of tie rods and/or strapping elements, it is also possible to quantify the effective perimeter area of the support walls, which receive and make power transmission (Ferreira 2010).

The total perimeter based on which coverage will be reduced due to the proximity of the eaves openings, since the masonry panels overlying the openings having a geometric ratio L/H can not guarantee the transmission of load (Fig. 16) (Vicente 2008).

If the perimeter is low or very low, due to the presence of gaps along the eaves, the impulsive nature of the coverage is of course compounded (this aspect does not define the class of vulnerability, only aids in assessing impulsivity of the roof) (Ferreira 2010).

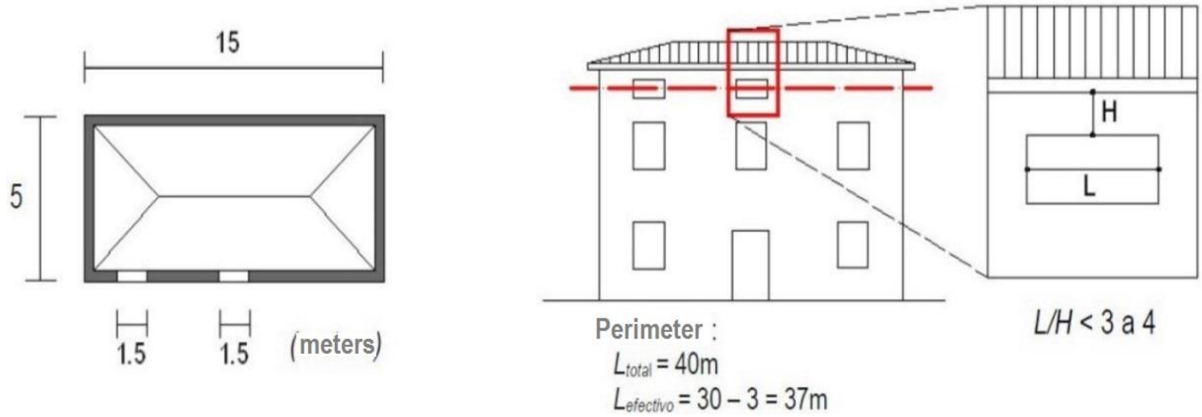


Fig. 16: Evaluation of the impulsive nature of the coverage (Ferreira 2010)

The vulnerability of this class parameter are defined as shown in Table 20. Fig. 17 (Ferreira 2010) is the most common structural types and classifies them as the impulsive nature, assisting the selection of the class of vulnerability in this parameter for identifying the structural typology. It is also expected in this parameter worsening classification of roofing depending on their condition (Ferreira 2010).

In the last decades, many in-situ post-earthquake investigations of the damages occurred in the masonry buildings have proven the devastators effect of the intrusive mixture between both materials of RC and historical masonry. Therefore, the exception of existing a coverage in reinforced concrete structure which is classified as Class A or B for this parameter, if it combined with a masonry poor quality, classified according to the Table 6 class C or D, the class that should be assigned in this parameter will be D (Ferreira 2010).

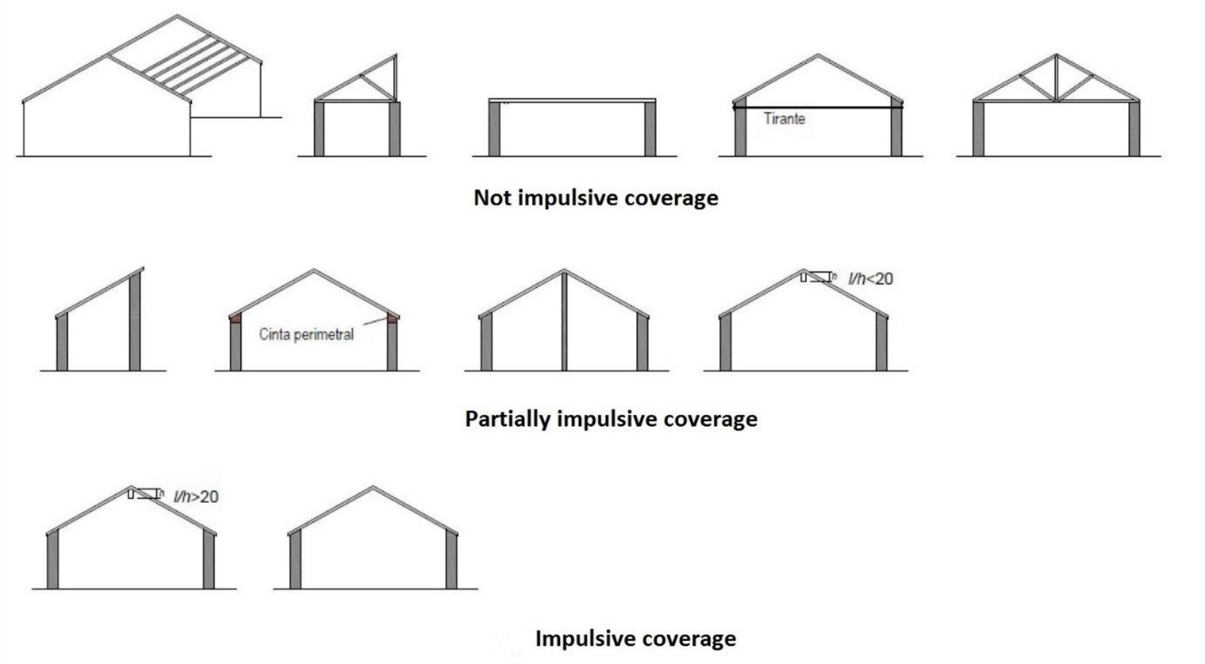


Fig. 17: Types of roofs and their classification regarding the impulsive nature (Ferreira 2010)

Table 20: Vulnerability classes definition for the parameter P11 (Ferreira 2010)

Classification	Perimetral cincture	Tie rod	Class	State of preservation	
				Bad	Terrible
Not impulsive	1 (yes)	1-0	A	B	C
	1-0	1	A	B	C
	0 (no)	0	B	C	D
Partially impulsive	1	1-0	B	C	D
	1-0	1	B	C	D
	0	0	C	D	D
Impulsive	1	1-0	C	D	D
	1-0	1	C	D	D
	0	0	D	-	-

b) Application

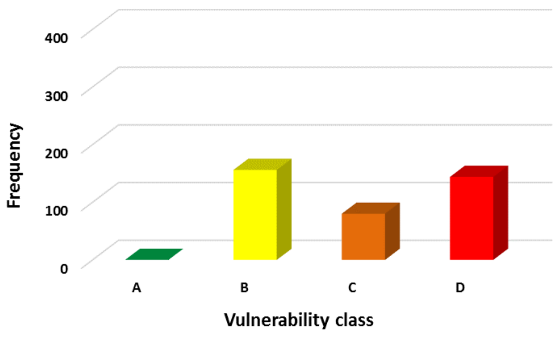
In the CTC data survey, the roof system is partially informed where indicated only its state of degradation. In this case, the supplementary information about the shape and the possible impulsion of the roof system are completed from the land use plans of the old town of Annaba city (DUC 2006). Moreover, although the degradation state of the roof system is partially detailed in the second part of the diagnosis of the CTC survey, however, we tried to classify the existing information as following (Table 21):

Table 21: Vulnerability classes of the parameter P11 according to CTC data survey

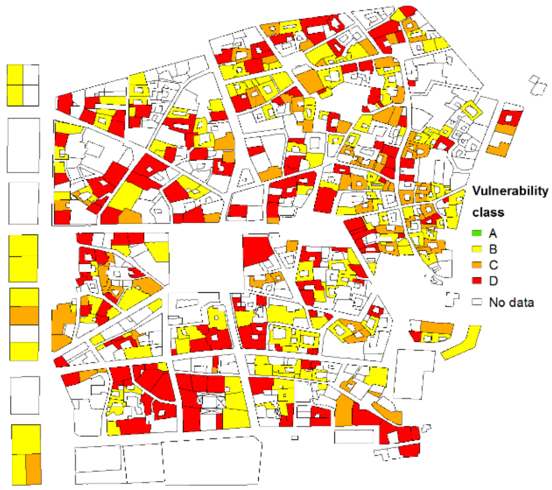
Classification	According to CTC data

Slight to moderate	- No degradation, Infiltration of water and traces of moisture - Moisture derived from infiltration
Heavy to very heavy	- Rotting head of beams - Movement of the ridge beam
Mediocre	- Leakage on the cover, Movement of the ridge beam - Rotting head of beams, Moisture derived from infiltration

Fig. 18 presents the results of the parameter P11 (Roof system) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city based on the descriptions presented in Table 21 and using additional documents (DUC 2006).



a) frequnecy



b) spatial distribution

Fig. 18: Roof system - P11-

Parameter P12: Intervention process

a) Definition

The observation and study of damage caused by seismic action in masonry buildings and particularly in the earthquake of Umbria-Marche in 1997, was shown the disastrous effect of the renovation actions especially using concrete solutions (solid and lightened) on masonry constructions of poor quality and poor execution. Therefore, this parameter accounts as far as possible anomalies at constructions level, which we observed in our society. Among these anomalies, one can quote the adjustments on the original structure (balcony transformed on a room, room transformed on a water storage zone, suppression of walls, adding RC columns …). These modifications cause a change in the center of mass, which affect the value of the seismic effort applied to the structure, constantly result in a deterioration in the response of the structure (Boukri and Bensaibi 2008, Djaalali *et al.* 2012).

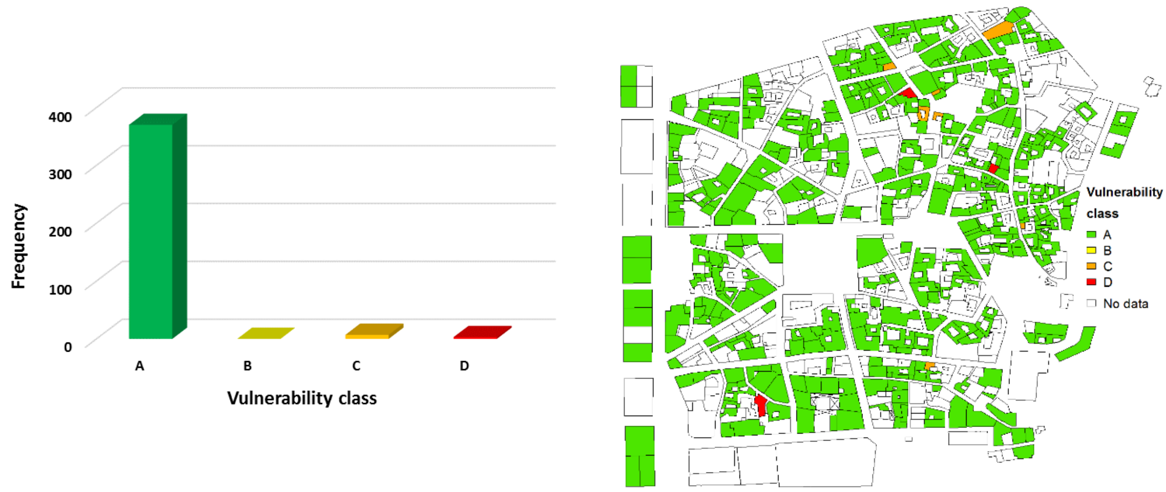
b) Application

The interventions process occurred on the masonry buildings of the old town are generally intrusive modifications. The vulnerability classes of this parameter are described on the basis of the interventions type stated in the first part of diagnosis (CTC survey), and inspired from the quality of the masonry walls (Table 6) especially for the case where RC elements are connected (see Table 22).

Table 22: Vulnerability classes of the parameter P12 according to CTC data survey

Vulnerability class	Classification	According to CTC data
A	Slight intervention	None, simple reparation or low to moderate rehabilitation process.
B	Moderate intervention	Adding RC column or shear walls connected to a good to moderate quality of masonry walls.
C	Heavy intervention	Adding RC column or shear walls connected to poor quality of masonry walls.
D	Mediocre intervention	Deleting elements, Elevation or aggrandizement actions.

Fig. 19 presents the results of the parameter P12 (Intervention process) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the CTC data survey and some in-situ investigations.



a) frequncy b) spatial distribution

Fig. 19: Intervention process - P12 -

Parameter P13: General state of preservation

a) Definition

This parameter is intended to assess the weaknesses in the structure (walls, floors and roofs), which may exacerbate damage that may result from the occurrence of an earthquake. The vulnerability classes are defined by the severity of structural anomalies of origin (may even be originated from a previous seismic action) that can trigger certain mechanisms for more precipitately (Ferreira 2010).

Table 23 identifies, class by class, problems and ways to increase substantially the risk of buildings being damaged, particularly highlighting the degree of cracking and deterioration of materials: cracks along the corners, shutdown of orthogonal walls, cracking by improper transmission of loads, bulging and deformation, signs of crushing, etc. (Ferreira 2010).

Table 23: Vulnerability classes definition for the parameter P13 (Ferreira 2010)

Class	Description
A	Masonry walls in good condition with no visible damage
B	Walls with small cracks (lower width to 0.5mm) not widespread. Signs of moisture that deteriorate the characteristics of masonry and lead to degradation of the coating of wood and masonry disaggregation. Cracks in the coating do not propagate to the support.
C	Walls with cracks opening about 2 to 3mm or showing cracking across the board (either may be the result a previous seismic action). Structures with a mediocre condition of the masonry walls, compromising their overall strength. Problems of severe deformation of the structure of a stairwell, deformations of floors, inclined cracks in interior partition walls, cracking at mid-span openings.
D	Walls with deterioration and even if not widespread severe cracking. Walls with reduced physical features and much degraded materials that show a serious decrease in resistance. Cracking in sensitive locations, such as near the corners (signs of disconnect between orthogonal walls). Damage introduced by impulses transmitted by the covers, bulging-resistant walls, cracking due to settlement of foundations. Slip half-timbered in relation to walls, rotting and degradation half-timbered along the walls. Signs of rotation of the walls and outside walls plumb.

b) Application

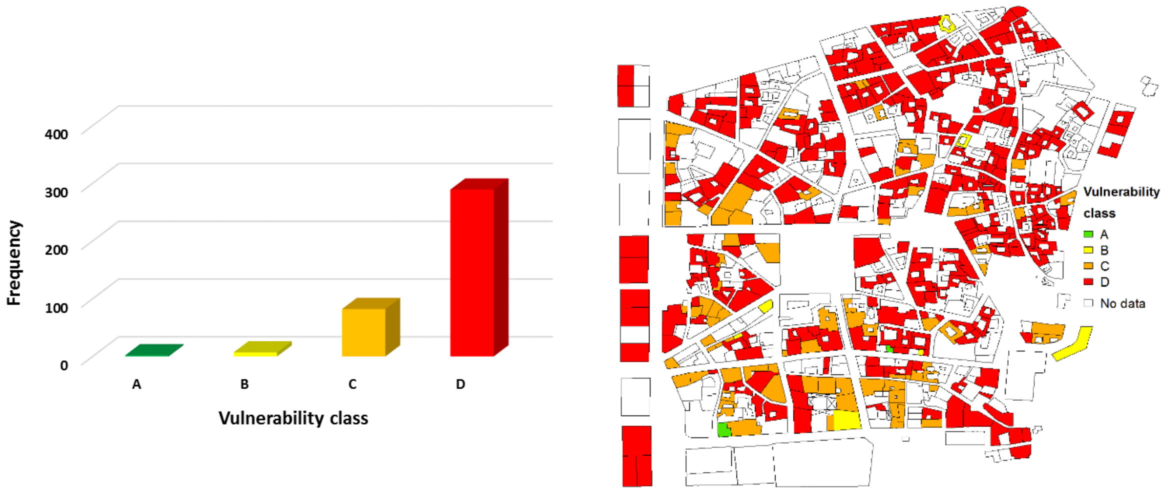
As already mentioned, the CTC engineering screener followed a qualitative approach based on an expert judgment of a visual diagnosis (from inside and outside of buildings) to classify them according to their degradation stat in view of assigning priorities for retrofitting interventions for their preservation. Additionally, they took into account all possible elements including even the secondary ones such as the stairs; covers, cladding, waterproofing, sewerage networks, etc., which have not any signification on the building behavior. However, in this regards, the CTC survey considered as a very useful data to define the vulnerability classes for the general state of preservation of the building. For understanding and an easy use, we simplified the definition of each class as presented in the Table 24:

Table 24: Vulnerability classes of the parameter P13 according to CTC data survey

Vulnerability class	Classification	According to CTC data
A	Good state	Building with compound structural or nonstructural elements of a good state.

B	Slightly degraded	Building with compound elements of a slight degradation state as defined in P2, P5, P11
C	Moderately degraded	Building with compound elements of a moderate degradation state as defined in P2, P5, P11
D	Highly degraded	Building with compound elements of a high degradation state as defined in P2, P5, P11

Fig. 20 presents the results of the parameter P13 (General state of preservation) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the adaptation defined in Table 24.



a) frequnecy b) spatial distribution

Fig. 20: General state of preservation - P13 -

Parameter P14: Non-structural elements

a) Definition

This parameter evaluates the effect of elements that are not part of the structural system, such as cornices, parapets, balconies or any other protruding element that is linked to the structure and whose solidarization weakens and increases the level of damage in structural elements (Ghislaine 2008) (see Table 25).

Table 25: Vulnerability classes definition for the parameter P14 (Ghislaine 2008)

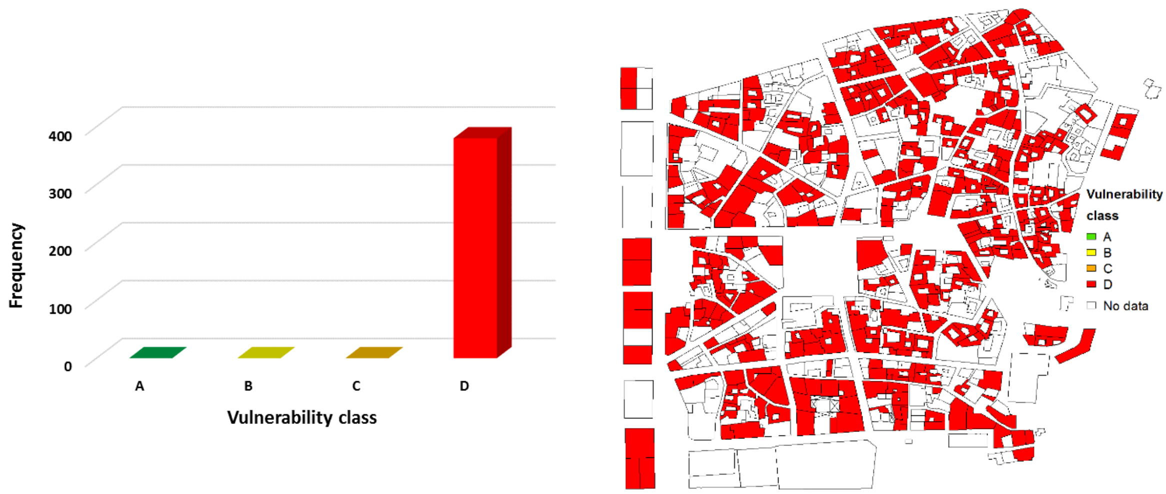
Vulnerability class	Description
A	- Buildings without dormant, appendices, objects, false ceilings.
B	- Buildings with dormant well connected to walls, chimneys of small size and moderate mass, false ceilings well connected - Buildings with balconies forming an integral part of the horizontal structures (floors)

C	-Buildings with external dormant or signboards of small size not well connected to walls and false ceilings of small size not connected or of large size well-connected
D	- Buildings with chimneys or other roof appendices bad attached to structure, parapets with bad execution or other heavy elements that may collapse in the event of earthquake - Buildings with balconies or other objects (service equipment ...) added after construction of the building and connected to the structure so summary - Buildings with false ceilings of large size and poorly connected

b) Application

For the present parameter, we took the same definition of each vulnerability class using the original GNDT method (Table 25) despite that the CTC data survey does not contain a good information corresponding to the non-structural elements. In this regard, based on certain detailed masonry buildings performed by the engineering Architects (from CFOS) and accounting the most events happened in the old town of Annaba city of falling down entire elements, which caused generally by the presence of cantilever balconies and the false ceilings elements of bad connection to the main structure, we suggested for this parameter P14, a vulnerability class "**D**" for entire masonry buildings stock.

Fig. 21 presents the results of the parameter P14 (Non-structural elements) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 25.



a) frequnecy b) spatial distribution

Fig. 21: Non-structural elements - P14 -

3.2 Conclusion:

Regarding the first applied methodology (modified GNDT II), new parameters were added, and too many adaptations have been performed on the most important parameters of the original version of GNDT II method according to the CTC data survey and in certain cases referring to additional documents. We considered all these facts as useful and enough for defining the vulnerability classes of each parameter that well revealing the conditions of the study area.

4. EMS98 concept: RISK-UE LM1 method

As already mentioned, in the current method, the main parameter is the typology of the buildings which are described in the original RISK-UE handbook (RISK-UE 2003). According to the latter, the typology of the buildings is performed considering the bearing masonry walls typology and the type of the diaphragm system which are well informed in the CTC data survey.

Moreover, the modifier parameters taken into account in the RISK-UE LM1 method that affect the behavior of the building are feed from the same data survey with a slight adaptation of certain parameters.

In the same way of the previous method, to minimizing the uncertainties of the final vulnerability index, two new parameters are added in the aim of taking into account the maximum available information having significant role to characterize the building' behavior.

4.1 Building typology

Based on the different typologies of masonry buildings defined in the original RISK-UE method, the Table 26 presents accordingly the typologies of both structural elements (load bearing wall and the diaphragm) existing in the CTC data survey.

Table 26: Selected data from the CTC survey

Load bearing elements		**Horizontal structure**	
Wall in Rubble stone	RS	Structure slab with wooden secondary elements	SS
Wall in Adobe	AD	Wooden slab connected to bearing walls with steel ties	W
Wall in Massive stone	MS	Masonry vault with steel ties	V
Wall in old brick	OB	Composite slab in steel and masonry	SM
RC shear wall and masonry wall	RM	Reinforce concrete slab	RC

According to the existing CTC data, in certain buildings two or more types of masonry typologies or horizontal structure are marked, in this case, for the identification of the RISK-UE typology, we selected the most informed and detailed in term of CTC data, which corresponding generally to the oldest typology and the most degraded. It was thus possible to define the distinct typologies of the masonry buildings stock under study. The definition of each typology of masonry building can be directly deduced according the Table 27.

Table 27: Masonry building typologies according to RISK-UE LM1 method

Availability in CTC data		Horizontal system				
		SS	**W**	**V**	**SM**	**RC**
Resisting	**RS**	M1.1	M3.1	M3.2	M3.3	M3.4
	AD	M2	M3.1	M3.2	M3.3	M3.4

	MS	M1.3	M3.1	M3.2	M3.3	M3.4
system	**OB**	M1.2	M3.1	M3.2	M3.3	M3.4
	RM	-	-	-	-	-

From the analyzed data, we obtained distinct typologies of the masonry buildings stock under study. Table 28 shows that the historical masonry buildings of the old town of Annaba city comprise nine typologies.

Table 28: Distribution of the building typologies according to RISK-UE guidelines

Typologies		**Description**	∑	**%**
Masonry	M1.1	Rubble stone	90	23.68
	M1.2	U Masonry (old bricks)	29	7.63
	M1.3	Massive stone	1	0.26
	M2	Adobe	61	16.05
	M3.1	Wooden slabs	43	11.32
	M3.2	Masonry vaults	40	10.53
	M3.3	Composite steel and masonry slabs	100	26.32
	M3.4	Reinforced concrete slabs	16	4.21
		Total	380	100

Fig. 22 presents the distribution buildings typologies based on the outcome obtained above based on the information of the CTC data survey.

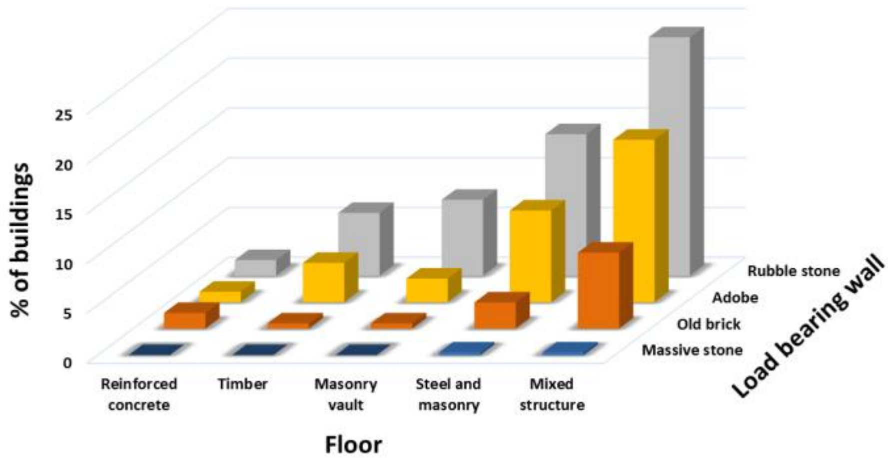


Fig. 22: Distribution of buildings according to their typologies, after (CTC 2010)

Keeping in mind that the probable vulnerability index value of each typology and its plausible bands as well as the maximum and the minimum limits, proposed by the RISK-UE are presented in Table III.12.

Subsequently, according to our database, the modifier parameters that are proposed by the current method are treated in their original form with the same weights (RISK-UE 2003). Two additional modifier parameters were added to take into account the degradation stat of resisting and the diaphragm systems. Table 29 lists all modifier parameters considering for the applied RISK-UE LM1 method

Table 29: Scores for the vulnerability factors V_m

N°	Typologies	ΔV_m
1	State of preservation	-0.04 to +0.04
2	Number of floors	-0.02 to +0.06
3	Degradation state of resisting system	+0.02 to +0.08
4	Degradation state of diaphragm system	0.00 to +0.06
5	Soft-story	+0.04
6	Plan Irregularity	+0.04
7	Vertical Irregularity	+0.02
8	Roof	+0.04
9	Retrofitting interventions	-0.08 +0.08
10	Aggregate building: position	-0.04 to +0.06
11	Aggregate building: elevation	-0.04 to +0.04
12	Soil Morphology	+0.02 to +0.04

4.2 Definition of the modifier parameters and its application using the CTC data survey:

Without going into deep details, the meaning of the most parameters are almost the same presented in the first part of this appendix for the GNDT level II method.

Parameter 1: State of preservation

This parameter is the same as the parameter P13 of the previous method (modified GNDT II), therefore based on the classification done in Table 24, the two possible cases of the state of preservation of the masonry buildings considered in the original RISK-UE LM1 method could be directly performed.

Table 30 shows the definition of the parameter P1 according to the original RISK-UE LM1 method and its application using the CTC data survey.

Table 30: Description of the Parameter 1 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Good maintenance	The buildings classified as slightly degraded or in a good state	-0.04
Bad maintenance	The buildings classified as moderately or highly degraded	+0.04

Fig. 23 presents the results of the modifier parameter P1 (State of preservation) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 30.

a) frequnecy b) spatial distribution

Fig. 23: State of preservation

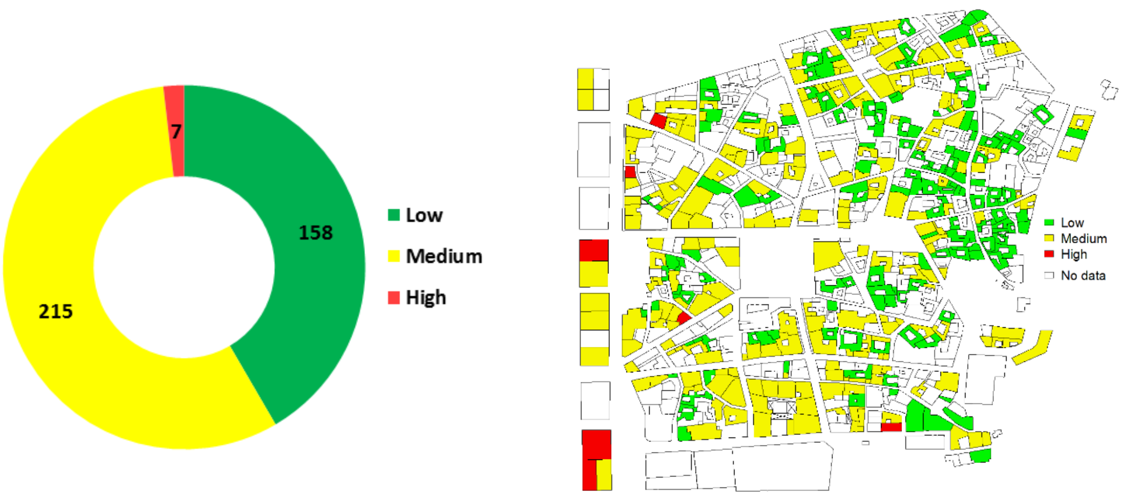
Parameter 2: Number of floors

As already indicated in the GNDT II method, the number of floor is an easy information, which is clearly informed in the documents provided by DUC (DUC 2006) and the CTC data survey wherein the possibility of existing an underground floor is also noted.

Table 31: Description of the Parameter 2 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Low (1 or 2)	1 or 2 levels	-0.02
Medium (3, 4 or 5)	3, 4 or 5 levels	+0.02
High (6 or more)	6 or more levels	+0.06

Fig. 24 presents the results of the modifier parameter P2 (Number of floors) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 31.



a) frequnecy b) spatial distribution

Fig. 24: Number of floors

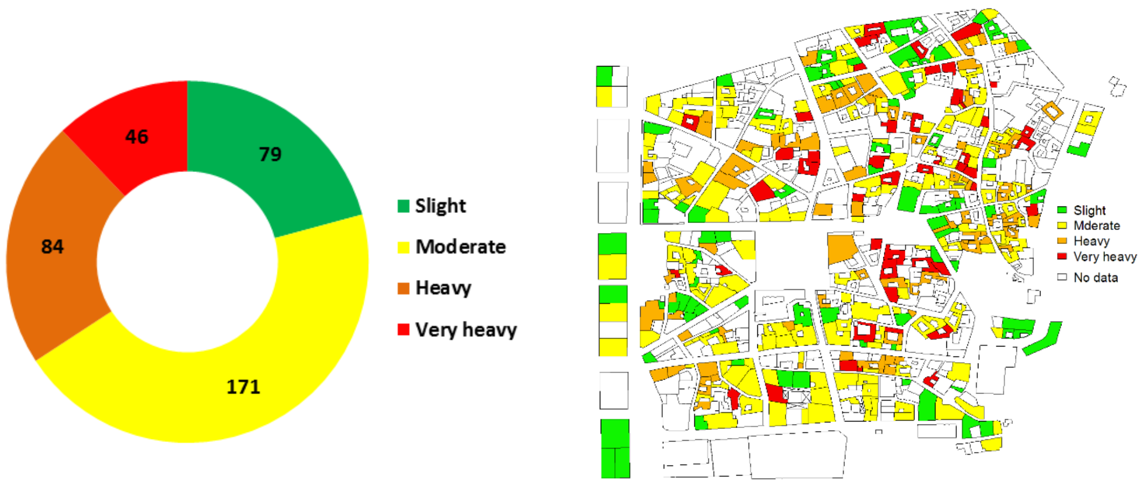
Parameter 3: Degradation state of resisting system

The additional degradation state parameter of resisting system is considered as an advantage in our methodology since the most parameters deemed in the majority of the developed approaches based only on the outside and street-walk information, in fact that the access to the interior of the buildings is often impossible. In contrary, the CTC data survey contains a very valuable inside information that led us to suggest the current parameter of degradation stat to exploit them, which is taken in direct accordance with that defined in the GNDT level II method (Table 4). However, for RISK-UE LM1 method, new weights of ΔV_m (RISK-UE 2003) were assigned to each stat (Table 32).

Table 32: Description of the Parameter 3 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Not estimated by method	Slight	+0.02
	Moderate	+0.04
	Heavy	+0.06
	Very heavy	+0.08

Fig. 25 presents the results of the modifier parameter P3 (Degradation state of resisting system) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 32.



a) frequnecy b) spatial distribution

Fig. 25: Degradation state of resisting system

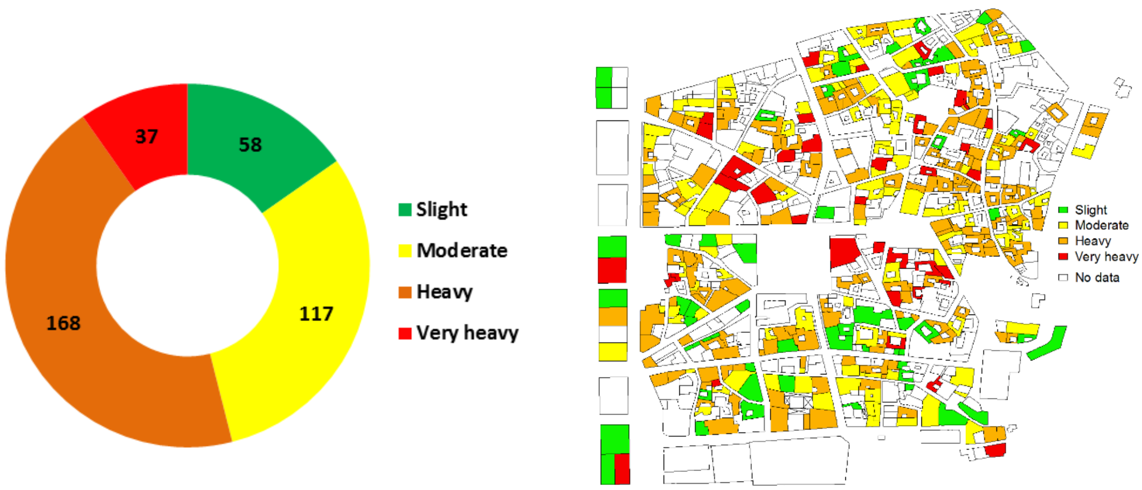
Parameter 4: Degradation state of diaphragm system

The additional parameter of degradation state of diaphragm system is our second advantageous parameter in our methodology for the same reason explained above. Therefore, keeping the same philosophy, the degradation state of the diaphragm system is taken in conformity with that defined in the GNDT level II method (Table 12), however also as can be shown in Table 33 new weights of ΔV_m (RISK-UE 2003) were allocated.

Table 33: Description of the Parameter 4 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Not estimated by method	Slight	0.00
	Moderate	+0.02
	Heavy	+0.04
	Very heavy	+0.06

Fig. 26 presents the results of the modifier parameter P4 (Degradation state of diaphragm system) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 33.



a) frequnecy b) spatial distribution

Fig. 26: Degradation state of diaphragm system

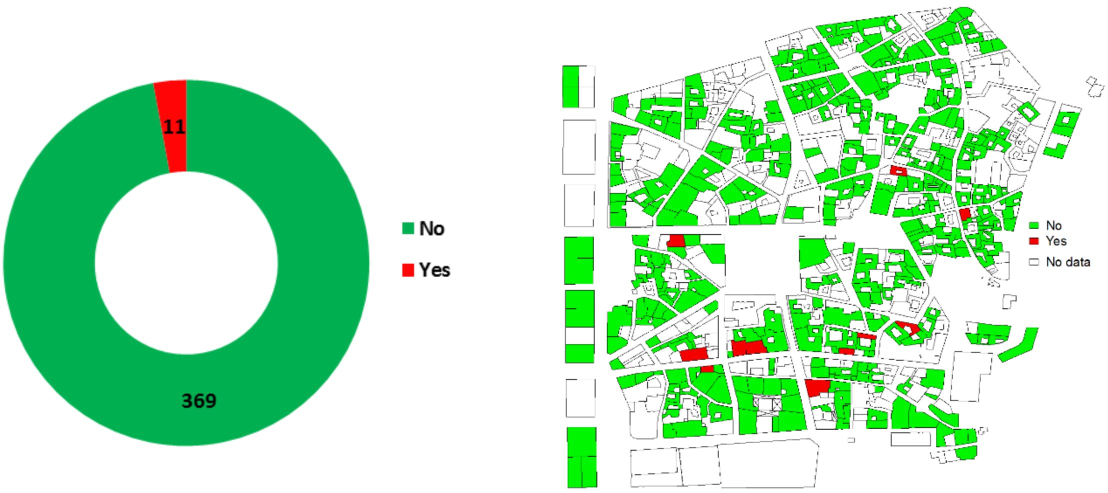
Parameter 5: Soft-story

In the RISK-UE LM1 method, an elementary known definition of a soft-story is taken for this parameter. Often, the ground floor of the buildings used for commercial business is considered as a soft-story. This parameter can be directly deduced from the buildings use information noted by the CTC screeners during their diagnosis.

Table 34: Description of the Parameter 5 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Demolition / Transparency	Demolition / Transparency	+0.04

Fig. 27 presents the results of the modifier parameter P5 (Soft-story) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 34.



a) frequnecy b) spatial distribution

Fig. 27: Soft-story

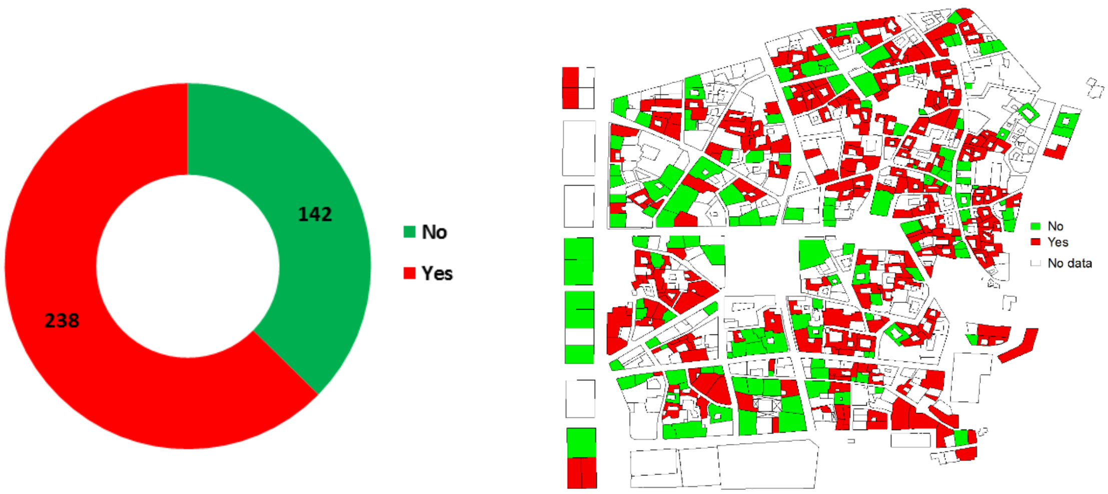
Parameter 6: Plan Irregularity

For a simple and rapid use, we related the definition of this parameter (Table 35) to the outcomes obtained from the application of the GNDT's parameter P9 described in Table 18. However, keeping the original RISK-UE LM1 weights of ΔV_m (RISK-UE 2003).

Table 35: Description of the Parameter 6 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
-	Vulnerability class "A" or "B"	0.00
	Vulnerability class "C" or "D"	+0.04

Fig. 28 presents the results of the modifier parameter P6 (Plan Irregularity) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 35.



a) frequnecy b) spatial distribution

Fig. 28: Plan Irregularity

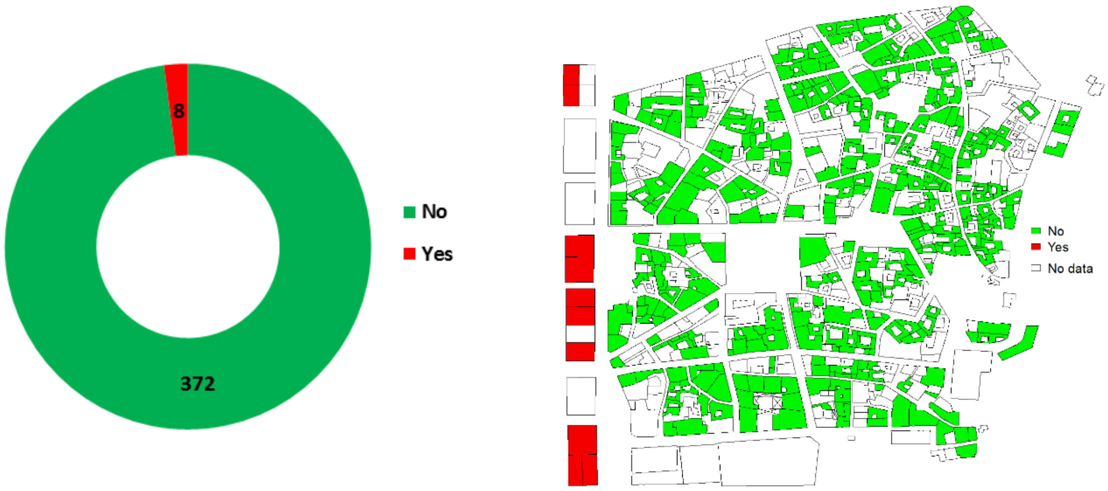
Parameter 7: Vertical Irregularity

As already mentioned in the first method, the majority of buildings have a good configuration in elevation, except some of them. In this regards, we took the same results found in the GNDT level II method with the original values of ΔV_m (RISK-UE 2003) as defined in the Table 36.

Table 36: Description of the Parameter 7 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
-	Vulnerability class "A" or "B"	0.00
	Vulnerability class "C" or "D"	+0.02

Fig. 29 presents the results of the modifier parameter P7 (Vertical Irregularity) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 36.



a) frequnecy b) spatial distribution

Fig. 29: Vertical Irregularity

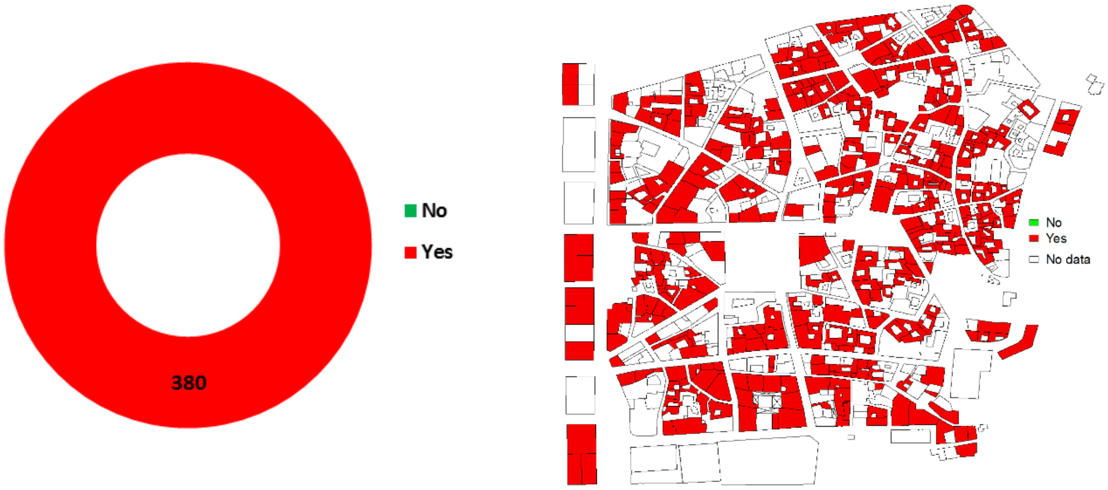
Parameter 8: Roof

The definition of the roof system in the tow applied methodologies is quite different. Moreover, the CTC data survey does not contain an accurate information corresponding to the roof system as defined in the RISK-UE LM1 method (RISK-UE 2003), however, we suggested that all the masonry buildings located in the old town of Annaba city suffered by an undesirable effect of the roofing system, thus we considered it has a disadvantage influence for all buildings.

Table 37: Description of the Parameter 8 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Roof weight + Roof Thrust Roof Connections	We considered a negative effect for all the buildings	+0.04

Fig. 30 presents the results of the modifier parameter P8 (Roof) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 37.



a) frequnecy b) spatial distribution

Fig. 30: Roof

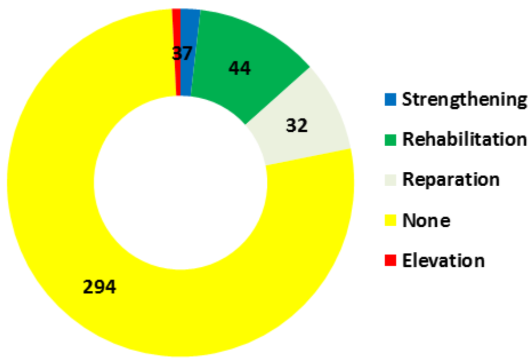
Parameter 9: Retrofitting interventions

After checking the DUC (direction of the urbanism and construction) and the DUCH (direction of urbanism construction and habitation) documents about the interventions that were occurred in the old town of Annaba city, we concluded that almost all the interventions have a negative effect on the building’s behavior such as the aggrandizement and elevation processes, where some compound elements are deleted or other added. These modifications affect the center of the mass, subsequently the effort applied on the structure. Regarding the rehabilitation process is considered in our case as a moderate reparation process in fact that no improvements of the building’s behavior have been shown. Whereas, the strengthening process even that generally has not been undertaken with rigorous technics against the seismic events, however it could be quite considered as an advantage for the building’s behavior.

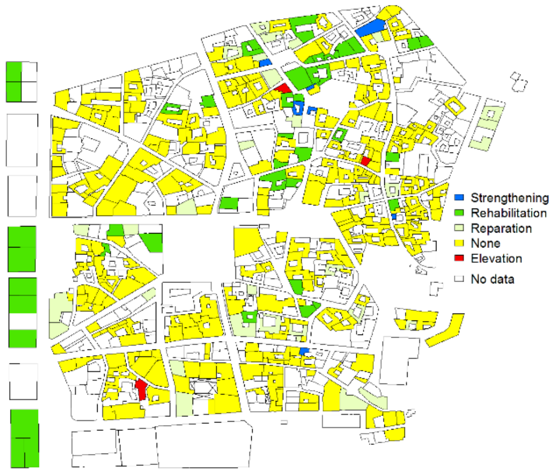
Table 38: Description of the Parameter 9 according to the CTC data survey

Description according to RISK-UE method	**Description according to CTC data survey**	ΔV_m
Retrofitting interventions	Elevation	0.08
	Aggrandizement	0.04
	None	0.00
	Reparation	0.00
	Strengthen	-0.04
	Rehabilitation	-0.02

Fig. 31 presents the results of the modifier parameter P9 (Retrofitting interventions) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 38.



a) frequnecy



b) spatial distribution

Fig. 31: Retrofitting interventions

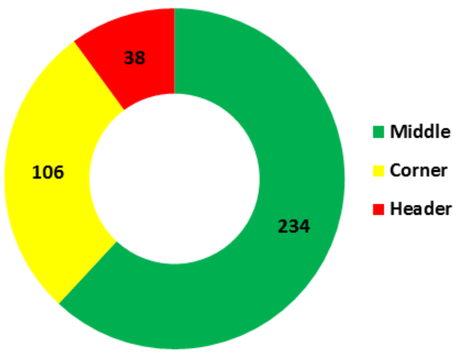
Parameter 10: Aggregate building: position

This parameter is the same exist in the modified GNDT level II method, so we took the same definition presented in Table 17 with the inherent weights of RISK-UE method (Table 39).

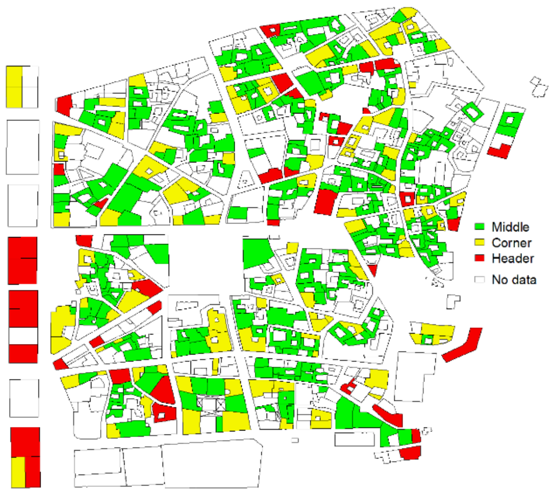
Table 39: Description of the Parameter 10 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Middle	Limited by three buildings or by two buildings in parallel sides	-0.04
Corner	Limited by two buildings in the two opposite sides	+0.04
Header	Limited by one building in one side	+0.06

Fig. 32 presents the results of the modifier parameter P10 (Aggregate building: position) in term of frequency and spatial distribution for the study area according to the Table 39.



a) frequnecy



b) spatial distribution

Fig. 32: Aggregate building: position

Parameter 11: Aggregate building: elevation

The classification of the buildings in elevation in their aggregates stock is defined in terms of the difference in height between the adjacent buildings as shown in Fig. 33 (Lantada *et al.* 2010).

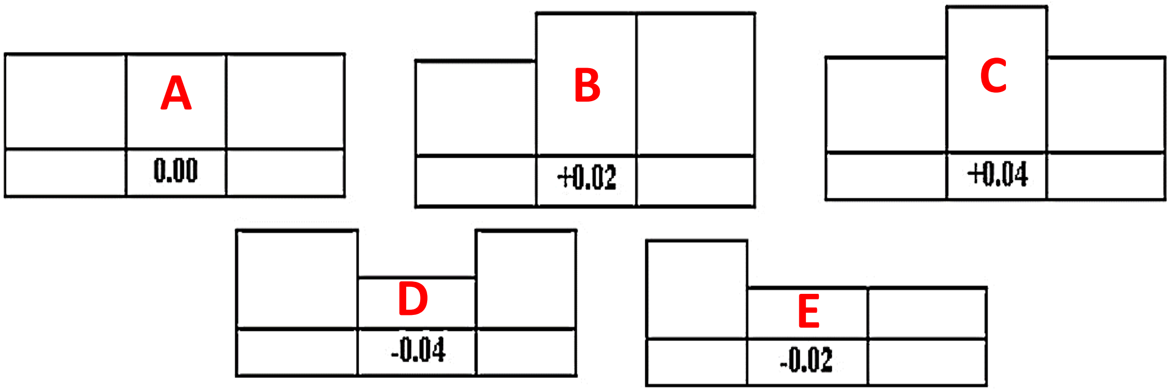
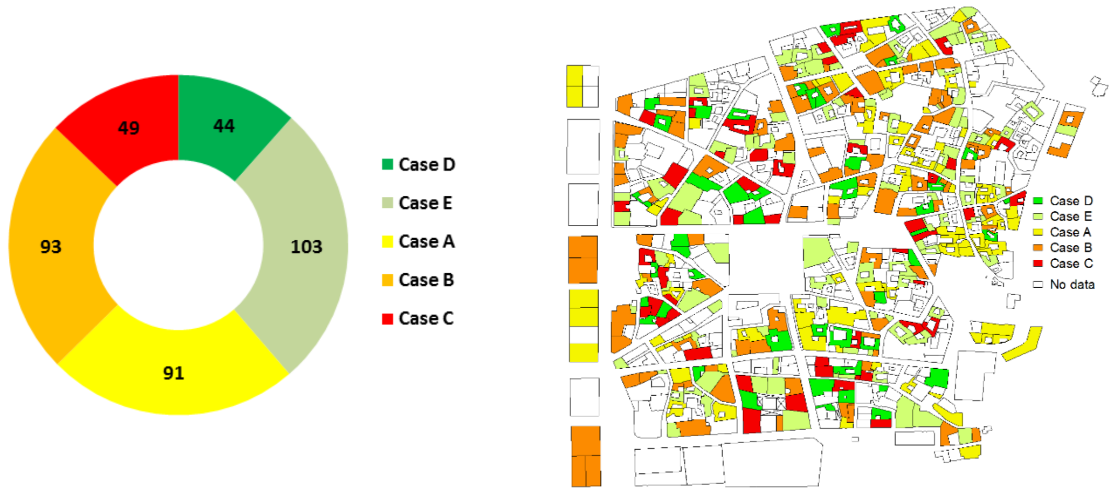


Fig. 33: Location modifiers for each building according to the difference between its height and the height of the two adjacent buildings (Lantada *et al*. 2010)

Table 40: Description of the Parameter 11 according to the CTC data survey

Description according to RISK-UE method	**Description according to CTC data survey**	ΔV_m
Staggered floors	Staggered floors	+0.02
Buildings of different height	Case D	-0.04
	Case E	-0.02
	Case A	0.00
	Case B	+0.02
	Case C	+0.04

Fig. 34 presents the results of the modifier parameter P11 (Aggregate building: elevation) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 40.



a) frequnecy

b) spatial distribution

Fig. 34: Aggregate building: elevation

Parameter 12: Soil Morphology

Due to the lack of reliable information about the morphology of the soil for the case of the old town of Annaba city, we referred in this parameter only to the degree of the slop. Therefore the description of this last parameter according to the CTC data survey is illustrated in Table 41.

Table 41: Description of the Parameter 12 according to the CTC data survey

Description according to RISK-UE method	Description according to CTC data survey	ΔV_m
Slope	Slight to moderate slope	+0.02
Cliff	High slope	+0.04

Fig. 35 presents the results of the modifier parameter P12 (Soil Morphology) in term of frequency and spatial distribution for the historical masonry buildings of the old town of Annaba city according to the Table 41.

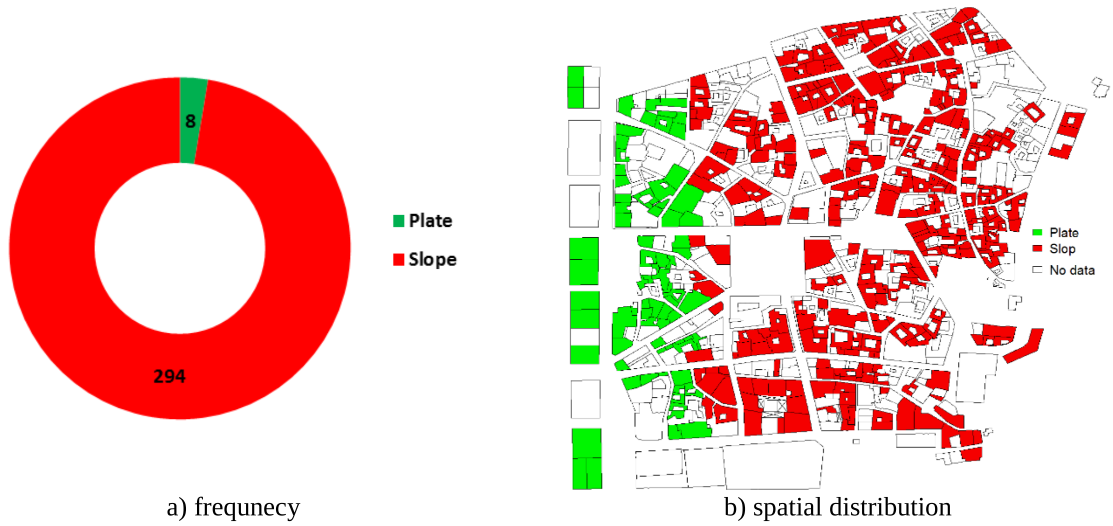


Fig. 35: Soil Morphology

4.3 Conclusion

Almost the definition of each parameter of the original RISK-UE LM1 method is available in the CTC data survey. However, for the structural parameter which was complicated to perform, is replaced by the degradation state of the structural elements that well informed in the existing data survey.

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