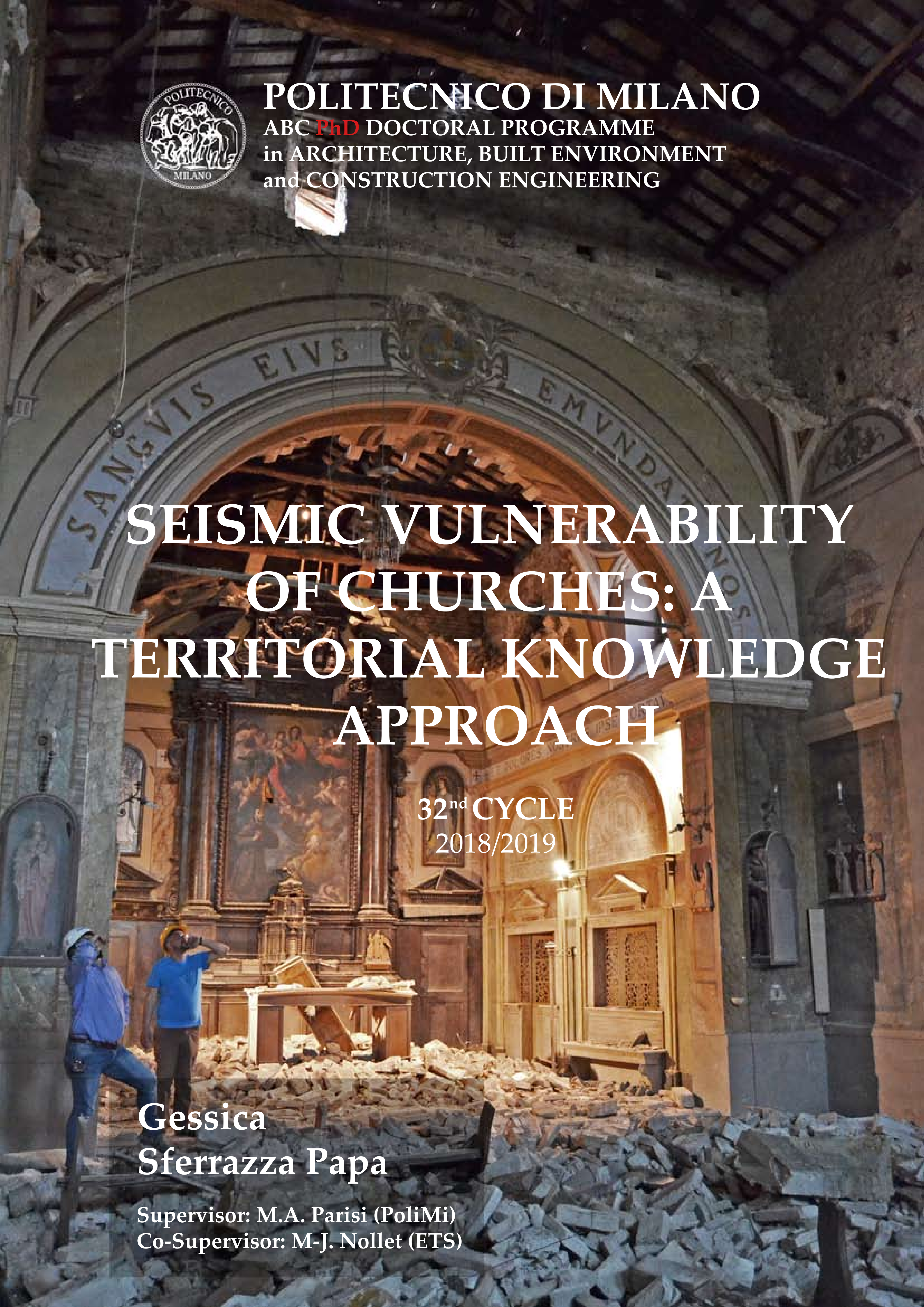

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SEISMIC VULNERABILITY OF CHURCHES: A TERRITORIAL KNOWLEDGE APPROACH

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IMAGE OF THE COVER
SS. Crocifisso church, Tolentino (MC), Italy.
Photo's rights: Gessica Sferrazza Papa
Taken during the post-earthquake damage surveys in the Spring 2017.

The cover is dedicated to all the people that lost a fragment of the Cultural Heritage of their towns due to the earthquake.

To
my family
Jacopo
and Elena

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LA VULNERABILITÀ SISMICA DELLE CHIESE: UN APPROCCIO DI CONOSCENZA TERRITORIALE

Sommario

Questo studio si rivolge alla vulnerabilità sismica delle chiese, proponendo un approccio territoriale per la sua valutazione.

In Italia, negli ultimi decenni si sono succeduti una serie di importanti terremoti, che hanno messo in luce il rischio a cui è esposto il patrimonio culturale. In quest'ambito si è evidenziata particolarmente la vulnerabilità delle chiese, dovuta alle caratteristiche strutturali intrinseche di queste costruzioni. Studi sistematici sulla risposta sismica degli edifici in muratura storica e in particolare delle chiese sono stati sviluppati a partire dal terremoto del Friuli del 1976. Tali studi hanno portato, nel tempo, ad elaborare metodi per la valutazione della vulnerabilità sismica attraverso l'identificazione dei macroelementi che compongono questi edifici e dei corrispondenti meccanismi di danno.

L'approccio qui proposto si caratterizza per l'identificazione delle cosiddette specificità territoriali, ovvero aspetti che influiscono sulla risposta sismica e che è auspicabile considerare nell'ambito di un'analisi della vulnerabilità, e alla integrazione di tali conoscenze con le attuali procedure di valutazione basate principalmente sul riconoscimento del modello di danno. Esso porta ad un metodo per la valutazione della vulnerabilità sismica delle chiese che è specificamente legato al territorio e alle caratteristiche costruttive, mantenendo come riferimento e incrementando la procedura di base che è attualmente disponibile.

Sono esaminate, quindi, alcune chiese provenienti da due aree geografiche italiane (Centro-Italia e Lombardia orientale) – dove si riscontrano una diversa consapevolezza della sismicità, diversità di materiali da costruzione e distinte tecniche costruttive - e un caso specifico di chiesa-fortezza, l'Arabo-Normanna, nella Sicilia orientale. Da questa indagine, viene proposto un approccio territoriale che porta all'elaborazione di una scheda di rilievo e valutazione (TSK-Form) che considera la specificità territoriale per indagini sulla vulnerabilità.

Infine, l'approccio proposto viene trasferito e testato a Montreal (Québec), dove la diversa cultura costruttiva e la locale consapevolezza della sismicità hanno portato a costruire chiese con caratteristiche diverse da quelle italiane. L'applicazione dell'approccio territoriale proposto ha dimostrato la capacità di interpretare la vulnerabilità delle chiese anche in questo contesto.

Le informazioni raccolte nelle operazioni di rilievo contribuiscono anche allo sviluppo di una base di conoscenze sulla vulnerabilità sismica delle chiese dei territori considerati. Queste informazioni vengono, quindi, catalogate in uno spazio di lavoro GIS, di cui è fornito un esempio alla fine della tesi. L'utilizzo di questo software, sia in fase di sviluppo della ricerca che in fase di raccolta finale delle conoscenze delle specificità territoriali, riflette il carattere di flessibilità e adattabilità proprio di questa ricerca.
L'adattamento della metodologia al contesto canadese ha aperto nuovi percorsi di indagine, ora in corso, sulle tipologie di altre chiese locali, al di là della tipologia neoromanica affrontata in questo studio.

Parole chiave: *Vulnerabilità sismica delle chiese, Prevenzione dei danni sismici, Terremoto della Pianura Padana Emiliana (2012), Terremoto del Centro Italia (2016), Québec, cultura costruttiva locale, specificità territoriali, TSK-Form.*

THE SEISMIC VULNERABILITY OF CHURCHES: A TERRITORIAL APPROACH

Abstract

This study focuses on the seismic vulnerability of churches, proposing a territorial approach for its assessment.

Several earthquakes have occurred in Italy along the years, giving evidence of the risk to which the Cultural Heritage building stock is exposed. During these devastating natural disasters, churches have shown their seismic vulnerability, mostly due to their intrinsic structural characteristics. Systematic studies have been performed since the 1976 Friuli earthquake on masonry structures, with particular emphasis on heritage buildings. They have provided methods for vulnerability assessment, identification of macro-elements and the corresponding damage mechanisms, and for damage classification.

The approach proposed here is the identification of the so-called territorial specificities, that is, local construction characteristics to be considered within a vulnerability analysis to complement the current assessment procedures based mainly on damage pattern recognition. It leads to a method for the seismic vulnerability assessment for churches that is specifically related to the territory and to the building characteristics, keeping as reference and advancing the basic procedure that is currently available.

Some churches from two Italian geographical areas (Central Italy and Eastern Lombardy), with different awareness of seismicity, construction materials and techniques, and a specific case of church-fortress, the Arabic-Norman, in Eastern Sicily, are examined. From this investigation, a territorial approach is proposed leading to the elaboration of a Territorial Specificity Knowledge Form (TSK-Form) for vulnerability surveys. Finally, the proposed approach is transferred and tested in Montreal (Québec), where the different construction culture and seismicity awareness have led to build churches with features dissimilar from the Italian ones. The application of the territorial

approach proposed for the seismic vulnerability assessment of churches has shown the capability to interpret the vulnerability of churches also in this context.

The collected information also contributes to the development of a knowledge base on the seismic vulnerability of churches from the considered territories; this information is then formalized in a GIS workspace; an example is provided at the end of the thesis. This tool reflects the character of flexibility and adaptability of this research.
The adaptation of the methodology to the Canadian context has opened new paths of investigation, now in progress, on local church typologies, beyond the néo-roman typology investigated in this work.

Keywords: *Vulnerability of churches, Seismic damage prevention, 2012 Pianura Padana Emiliana earthquake, 2016 Central Italy earthquake, Québec, construction culture, territorial specificities, TSK-Form.*

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INTRODUCTION

CONTEXT AND CRITICAL ISSUES

Earthquakes are one of the adverse natural phenomena which threaten entire territories and for such reason, seismic prone areas often develop resilience capability. Nevertheless, when part of the built environment is lost, tangible and intangible values are affected (Grandori and Benedetti 1973; Throsby 1999). The recent seismic episodes occurred in Italy, such as the earthquakes of L'Aquila, 2009, Pianura Padana Emilia, 2012, and Central Italy, 2016, have once more reconfirmed the seismic vulnerability of historic masonry buildings, of architectural heritage and in particular of churches. The *vulnerability* is defined as the inclination of a structure to suffer damage of a specific level for a seismic event of a given intensity, according to (Protezione Civile 2020); other definitions may be found in (Dowrick 2003, Coburn and Spence 2002)

In the Italian territory, churches constitute a consistent part of the cultural heritage building stock, approximately 80% (Podestà et al. 2005). Beyond their religious function, they are landmarks of cities and landscapes and represent the shrine of artefacts in terms of cultural traditions, of techniques of construction, and a point of reference and aggregation for the community in everyday life. For such reasons, the preservation of these cultural heritage assets becomes a cultural duty. In this context, the comprehension of the seismic vulnerability of churches contributes to set strategies for prevention.

In geographical areas highly prone to earthquakes, the seismic awareness may motivate specific provisions and has marked the construction traditions. In this dissertation, two types of awareness are distinguished. They may be defined as follows:

- *historical seismic awareness* when some construction solutions devised to face the seismicity of the territory have entered the construction practice in the past. Some examples can be found in (Marino 2000, D’Antonio 2018).
- *modern seismic awareness* when it is reflected in construction practice with interventions performed with modern techniques according to design codes. Such awareness indirectly affects the vulnerability of historical masonry churches. Indeed, the influence of modern interventions on the response of historical buildings was remarked in particular after important seismic events of

the last decades, such as the Umbria-Marche earthquake in 1997 (Tobriner, et al. 1997; Lagomarsino 1998; Modena and Binda 2009), the Molise earthquake in 2002 (Lagomarsino and Podestà 2004a), and the Central Italy earthquake of 2016 (e.g. Marotta et al. 2017a, Parisi et al. 2018; Penna et al. 2019, Canuti et al. 2019, Cescatti et al. 2020). Seismic strengthening interventions on historical buildings along the years have been regulated by structural design codes and guided by charters of Restoration (ICOMOS 2019). Interventions considered, in a specific period, innovative and effective have, in some cases, demonstrated with time not to satisfy expectations, or to be unsuitable for particular building conditions. This is the case, for instance, of strengthening and repairing interventions that significantly increase local mass and stiffness, such as concrete slabs and deep ring beams, too different and, therefore, not compatible with many weak masonry elements. A building inherited from the past, therefore, is the result of different time-related approaches and of the interventions carried out over the years.

Moreover, the specific history of geographical areas, as well as the availability of certain construction materials, has motivated the development of a construction practice typical of the place. As an example, to clarify this aspect, a medieval church in Central Italy is not built in the same way as a medieval church in Lombardy. They are constructed with different materials, set up according to the local construction knowledge, and, possibly, according to the seismic awareness of the area. Consequently, the structural response of the two churches, when subjected to an earthquake, will be different. This awareness of the differences in construction practice from one territory to another distinguishes the 1990s, when manuals showing a regional attention to the local construction practice were written (Giovanetti 1992; Giuffré 1993; Giovanetti 1997; Giuffré and Carocci 1997; Giuffré et al. 1999; Gurrieri 1999). In particular, after the 1997 Umbria-Marche earthquake, in 2000, the Regional Authority of Marche published the first version of a Code of Practice for interventions of seismic retrofitting in the restoration of the cultural heritage, which was later reviewed including a description of the interventions carried out according to the first version of the same code (Doglioni and Mazzotti 2007). Another example of attention to the local construction characteristics is the Atlas of the types of walls in Northern Italy, elaborated by the Ministry of Cultural Heritage (Mannoni 1995).

The long Italian seismic history, characterized by frequent strong earthquakes over the whole territory, has boosted specific vulnerability and damage assessment studies for churches, leading to the definition of the concepts of *macro-element* and *mechanism* (Doglioni et al. 1994). For this building category, the 1976 Friuli earthquake is recognized as the starting point of the studies performed in the following years and until today (e.g. Lagomarsino and Podestà 2004a; 2004b; 2004c; Lagomarsino 2012; Sorrentino et al. 2014). Methodologies for the seismic vulnerability assessment of churches have been elaborated and can be generally divided into rapid and detailed procedures. A rapid method, which will be cited in the following as *original procedure*, is described in the Guidelines for the assessment and reduction of the seismic risk of the cultural heritage building stock (MiBAC 2011), proposed in agreement with the Italian Construction Building Code (NTC 2008). The assessment is performed by attributing to each limit mechanism, among a pre-set list of 28, a value based on the presence of both seismic provisions and vulnerability–inducing elements. The total sum, conveniently weighted, provides an index of vulnerability (i_v) for the asset.

On the one hand, this procedure gives synthetic information on the relative seismic vulnerability of the church and, in a seismic risk perspective, contributes to set priorities for strategies at a large scale. On the other hand, the procedure of assigning a coefficient of vulnerability for the assessment of a church is rather subjective (De Matteis et al. 2017) and does not recognize its specificities as historical building, which are the result of the construction culture and history of its territory.

With the intention to reduce uncertainties and judgment subjectivity, an automatic form for the damage and vulnerability assessment called MaCHRO (Masonry CHurches Reconnaissance Operational Form) form was developed in 2017 (De Matteis et al. 2017). It combines the detailed procedure proposed for the damage and vulnerability assessment in (Podestà et al. 2005) and the rapid vulnerability assessment procedure recommended in (MiBAC 2011). Such a procedure reduces risks of inconsistency in the evaluation.

Another limitation of the original procedure is the absence of flexibility in the survey. There is no possibility for including mechanisms not previously identified and listed among the 28 mechanisms. This aspect is extremely evident when the methodology of seismic vulnerability assessment is transferred to a different environment, as this thesis will show for the case of Canada. Such a problem has also been discussed in the literature, in occasion of various earthquakes: in the Azores earthquake, 1998 (Magalhães et al. 2012), in Chile, for the Maule earthquake, 2010, (D'Ayala and Benzoni 2012) and in the Philippines, the Bohol and the Typhoon Haiyan earthquakes, 2013, (D'Ayala et al. 2016), as well as in New Zealand, for the Canterbury earthquake, 2010-2011, (Leite et al. 2013; Lagomarsino et al. 2019).

This need for flexibility and adaptability may be perceived also over the Italian territory. As chapter 1 will show, since its first elaboration and implementation, the list of possible mechanisms passed from 16 to 18, and then to 28, and on some occasions, has shown to still be unsatisfactory to describe some existing conditions. These last considerations are shared by the research community, as mentioned in a recent publication (Lagomarsino et al. 2019) that provides an answer to such issues by elaborating a specific vulnerability model calibrated on the data acquired through the application of the CAF-D (Church Assessment Form-Damage) in New Zealand (Goded et al. 2018). On their side, Marotta et al. (2018) develop regression models for the observed mechanisms in the new Zealander churches affected by the 2011 Canterbury earthquake including vulnerability modifiers, calibrated through statistical procedures.

The critical issues outlined in the thesis and synthetized in Figure 1 have motivated the development of an approach that refers to the territorial specificities for the seismic vulnerability assessment of churches, as proposed in this research project. The terms *territorial specificities* refer to those aspects typical of a territory, which are subjected to significant changes from one territory to another. Examples of territorial specificities that have influenced the construction practice are materials and techniques of construction, church typologies common in the area, history of local seismicity and its awareness, and cultural history. The proposed methodology focuses on the building conceived as a whole and leads to the formulation of a Territorial Specificity Knowledge Form (TSK-Form) for the assessment survey.

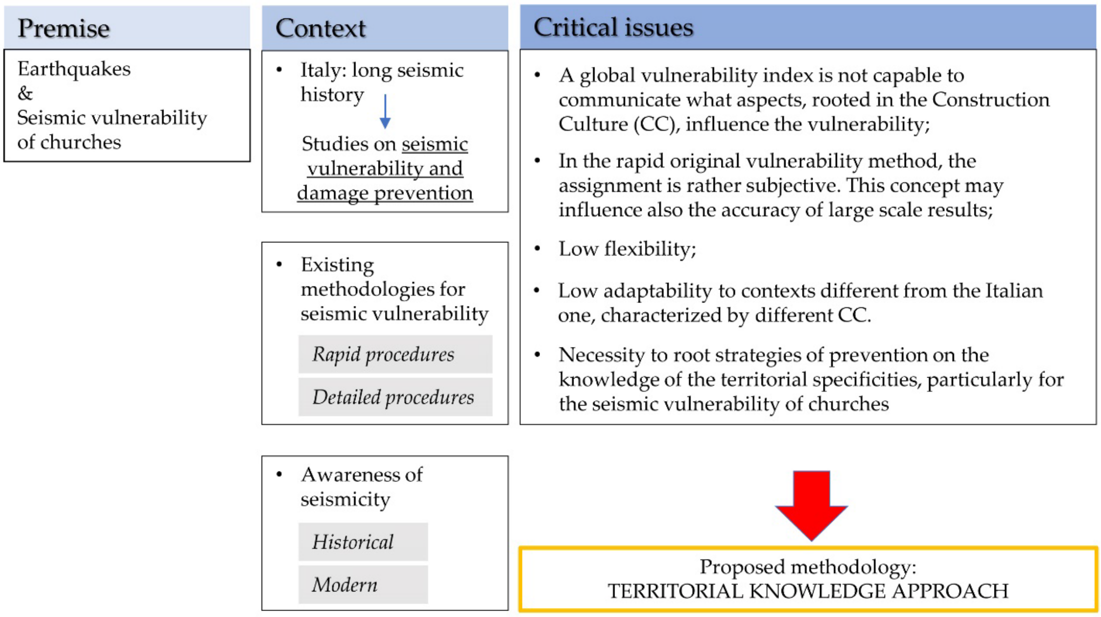


Figure 1 Synthesis of the principal aspects which motivated the development of the methodology.

OBJECTIVES AND METHODOLOGY

Considering the issues outlined above, the aim of this work is to contribute to the preservation of historical masonry churches from earthquake damage through the development of a methodology for seismic vulnerability assessment acknowledging territorial specificities. The approach resulting from this study is intended for use in strategies of prevention. The main objectives of this research may be summarized here:

- to develop a methodology for the seismic vulnerability assessment of churches, rooted on the knowledge of territorial specificities, having as main characteristics the flexibility and adaptability to different construction cultures;
- to create in people concerned an awareness of the seismic vulnerability of churches in relation to the territorial specificities; consequently, expressing a qualitative grade of vulnerability for the identified ones. These actions provide knowledge useful for decision-makers for damage prevention strategies;
- to provide examples of methodological practice by collecting territorial specificities for different territories. In this regard, a GIS workspace is used as a tool for making the information easier to be implemented and consulted. Such

tool would allow to adapt the collection of information to different construction conditions.

In order to reach these objectives, the following questions have guided the research path:

- How does seismic awareness in the territory influence the vulnerability of churches?
- How do territorial specificities influence the seismic vulnerability of churches?
- Is the proposed methodology adaptable to other contexts with different conditions?
- How can the "limit mechanism approach", 28 kinematic mechanisms, be updated in the light of territorial specificities?

As a result, flexibility and adaptability to different contexts appear as the most desirable characteristics for the approach to be developed. Final users should be the local Authorities, such as the Dioceses, the Superintendences in charge of cultural heritage protection, and the local administrations which operate in a specific geographical area.

The original mechanism-based procedure was developed for rapid investigations. In a less time-constrained condition, the seismic vulnerability assessment can be performed at different levels of detail depending on the specific situation. In some cases, geometrical and numerical models can be developed to understand the structure of the building and its seismic behavior as a basis for identifying the damage mechanisms most likely to develop. When common typologies or conditions are identified, considerations from a previously studied case can be extended to another, contributing to the process of knowledge acquisition on other churches. In other cases, the observation of damage in other contexts with similar conditions and the consultation of the literature become crucial to include all relevant aspects in the vulnerability assessment procedure: examples are the structural interventions executed in a certain way that have shown to be either ineffective or helpful in improving the seismic performance of the structure. Other post-earthquake analysis observations highlight the possible weakness of the building in cases where no intervention has been performed.

The main actions performed for the development of the methodology are:

- Consultation of archives (particularly the seismic Archive in the Superintendence of Brescia and the Diocese Archive of Montreal);
- Onsite damage surveys in Central Italy;
- Vulnerability assessments in damage prevention operations;
- Elaboration of numerical models to confirm assumptions on the structural behavior or for comparison with the occurred damage.

The investigation was performed starting from three Italian geographical areas (Central Italy, Eastern Lombardy, Eastern Sicily), different for seismic awareness and construction and cultural traditions. In each of these areas, a number of churches was selected to observe and to collect territorial aspects. The observations and the performed analyses lead to the elaboration of a methodology that was then tested in the Canadian context (Montreal, Québec), in occasion of a collaboration between the École de Technologie Supérieure (ETS) in Montreal and the Politecnico di Milano (POLIMI).

THESIS ORIGINALITY AND CONTRIBUTIONS

The main original contribution of this thesis is including the specific characteristics of the territory in the development of the seismic vulnerability assessment of churches. A historical building is put in relation with the territorial specificities, such as materials and construction techniques, local construction culture. Recognizing and gathering these aspects contributes also to create a local awareness of the seismic vulnerability related to the territory.

Another distinguishing contribution of this research is in the application to the case of Québec, a region with different cultural background. This part of the research is a test of the flexibility and applicability of the approach to another context that leads at the end of this research to the adaptation of the TSK-Form to the néo-roman typology identified among the churches of the Canadian territory. The adapted form differentiates itself from the Italian one for the territorial macro-elements and vulnerability modifiers. At the same time, the application provides a first dataset of knowledge on local churches that may be a reference for practitioners that operate in the field. In the Canadian context, the seismic vulnerability of churches is underestimated; additionally, projects of rehabilitation and installation of new functions related to a change of use in these historical buildings, which are just conceived as envelopes, have spread in the last years without regard to conservation. The vulnerability assessment procedure provides a sort of guideline to respond to these two issues. At the current state of knowledge, there are no recommendations nor guidelines that could constitute a reference when intervening on unreinforced masonry churches. This work may mark a beginning for the development of such guidelines.

LIMITATIONS AND OPEN RESEARCH SCENARIOS

The proposed approach leads to a qualitative evaluation of vulnerability. Levels of vulnerability are expressed as high, medium-high, medium, medium-low, low, based on heuristic knowledge. There is no intention to renounce to a quantitative evaluation, but a qualitative one was considered more apt due to the limited number of churches and territories treated so far. A quantitative evaluation can be performed at a later moment when there will be the availability of a considerable number of churches to constitute a significant statistical sample.

Another limit of the research is the inhomogeneity of the information between one geographical area and another. For example, in the case of Central Italy, the recent earthquake has determined the inaccessibility of archive data. Indeed, historical masonry buildings generally host archives and libraries that, in case of earthquake, are themselves subjected to damage.

The vulnerability assessment itself is performed compiling the TSK-Form, whose contents in terms of *territorial macro-elements* and *territorial vulnerability modifiers* (see section 6.4) are derived from the sample examined and it is not intended to be exhaustive.

The application to Québec provides a first exemplification and is proposed for one church category, the néo-roman; from this starting point, it can be adapted, extended, and modified according to other church typologies. Indeed, for the moment the proposed typological classification is based on the façade macro-element, but the use of another criterion of classification is not excluded. The present research has opened the path for a detailed investigation of another Québec church typology, the Conefroy, which is currently under study at ETS. The results from the study are intended to be compared and to be complementary to the outcomes of the present research on the néo-roman church typology, finally contributing to build a knowledge base on local typologies.

THESIS ORGANIZATION

The structure of this dissertation retraces the territorial approach. The first chapter describes the context of the research. The following chapters are organized according to geographical areas. The thesis considers two Italian regions (Chapters 3, 4), and a specific case, the church-fortress in Eastern Sicily (Chapter 5). This part supplies indications on territorial specificities. Chapter 6 details the approach based on the results from previous chapters. Finally, chapter 7 tests and validates the approach in Québec. Chapter 8 exemplifies how the information can be stored and set up in a GIS workspace. This tool was also used during the research to perform a multidisciplinary approach. Moreover, storing and managing the information with such a tool allows linking the information with the databases set and used by the Italian Civil Protection Agency and the Ministry of Cultural Heritage, or similar sources elsewhere, and it implies an easier accessibility to the information on churches.

Finally, general conclusions on the approach are drawn and recommendations made for the néo-roman church typology in the Québec context.

CHAPTER

1

THE SCIENTIFIC CONTEXT

This section retraces the origin of studies on the seismic vulnerability of churches that became the basis for the following work and current assessment methods. It presents codes, guidelines, and significant charters for restoration, which have set the modalities of intervention on the cultural heritage assets. The goal is to draw a picture of the context of where this study belongs.

1.1 A SHORT HISTORICAL EXCURSUS

When dealing with historical masonry buildings in relation to earthquakes, a premise is necessary: some important events have constituted a key point in the development of approaches of retrofitting interventions for the cultural heritage building stock. The historical buildings studied today are the result of changes and interventions over the years, each performed with reference to criteria and practice in use at the time.

Until the 18th century, interventions were performed following the principles of the *regola d'arte* ("rule of art"). Giuseppe Valadier titled his book *L'Architettura Pratica* (Practical Architecture) (Valadier 1828-1839), inaugurating in Italy the production of manuals devoted to good construction practice. Between 1828 and 1839, he wrote the first guide addressed to the art of construction and to interventions, following the principle of the "rule of art". This work included considerations on the damage in historical buildings and means to intervene on them. In particular, in section 20 entitled *Dalla maniera di osservare le lesioni negli edifizj, e metodo per rilevare le cause, e delle cautele per le riparazioni* (On the manner to observe cracks in buildings and method to detect causes, and cautions in repairing), Valadier emphasized the importance of an accurate survey of the existing cracks in order to understand their causes and to select the most suitable intervention. In the same book, Valadier continued highlighting seven main reasons responsible for damage in historical buildings. Among them, he mentioned the earthquakes, the age of the building, and the structural modifications over time. Finally, he put together the rules of good construction practice that, following the "rule of art", had demonstrated to be capable of ensuring the stability and the durability of buildings. Others researcher payed attention to the traditional seismic provision details in the historical architectures in the 20th century (e.g. Marino 2000), but in the same century, new materials, brought by modern technology, entered the field of restoration. Reinforced concrete soon received the favour of the experts. Calderini in (2008) explains the cultural and technical reasons lead the reinforced concrete becoming a largely used material in the restoration of monuments and in Fancelli et al (2003) the history of consolidation can be consulted. Here, an extract from the Charter of Athens, 1931, is reported: << […] the experts [...] approve the wise use of all modern technical resources, especially of reinforced concrete [...] These means of reinforcement should be concealed in order to avoid altering the appearance and character of the building […] >> (ICOMOS 2019).

The years that followed focused on defining guidelines for the practice of restoration. In Italy, Gustavo Giovannoni introduced the concept of “restauro scientifico” (scientific restoration) in the Carta Italiana del Restauro (Italian restoration chart), 1932, distinguishing several types of restoration, including the practice of consolidation. He was the first to support the inclusion of modern techniques and materials in the restoration field, promoting the principle of minimal intervention. In Giovannoni’s opinion, reinforced concrete had outstanding properties that could have provided a solution to numerous restoration problems. Giovannoni celebrated its simultaneous plasticity and rigidity, its capability to blend well with old materials, and its resistance to tension and bending. Nonetheless, he expressed some doubts on the long-lasting life cycle of concrete (Giovannoni 1931).

Even if the use of new techniques and materials was admitted in the Carta Italiana del Restauro, the Consiglio Superiore per le Antichità e le Belle Arti (Superior Council of Cultural Heritage) warned on the necessity to minimize the use of the new elements, paying attention not to change the structural behavior. In the same years Daniele Donghi, in his manual (Donghi 1935), admitted the use of reinforced concrete as suitable, as long as its function and aesthetics were well marked. Likewise, in the 10th article of the Carta di Venezia, 1964, Roberto Pane, Pietro Gazzola and Cesare Brandi confirmed the pertinence of using modern means, whose efficiency has been verified by scientific data for the consolidation of monuments, whenever traditional techniques proved to be inadequate (ICOMOS).

The new version of the Carta Italiana del Restauro, 1972, in its 7th article, introduces a list of admitted interventions. It comprises the addition of auxiliary parts with a static function, the reintegration of small parts, as historically established, and the modifications and new additions for static and conservation purposes in the internal structure, the substrate or the foundation. Once the intervention is completed, the appearance at the surface level must not be altered, neither chromatically nor materially. Additionally, it states that every action has to ensure the possibility of further interventions of restoration in the future.

In 1981, at the conference "Restoration of monuments and role of cement", the results of the use of concrete in the restoration until that time were extensively discussed (Carbonara 1981). That same year, at the National Congress of ASSIRCCO (Associazione Italiana Recupero e Consolidamento Costruzioni, Italian association for rehabilitation and consolidation of constructed facilities), Roberto Di Stefano affirmed that the restauration for consolidation had to be based on the historical investigation of the structure in its life, not only be performed as a mere structural intervention abstracted from its context (Di Stefano 1981). On its side, the Ministry of Cultural Heritage (Ministero Beni Culturali e Ambientali, BB. CC. AA.) issued the Circular No. 1032 on 18 July 1986, known as “Circolare Ballardini”, which pointed out that the << [...] interventions on monumental complexes were often conceived as static restructuration. They were carried out with a series of massive interventions that handle the culture of new materials with an ambiguous criterion, in particular of steel and reinforced concrete. Thus, the structural intervention strategy reshapes ancient buildings according to the resistant patterns of modern materials [...] The choice of widespread conservation must be carried out [...] in the seismic risk prevention >> (CMBA. 1986).

In the D.M. 24-1-1986 "Technical Standards for Construction in Seismic Zones", the concepts of *adeguamento sismico*, seismic strengthening, and *miglioramento sismico*, seismic improvement, were defined. The former is intended as << [...] the performance of operations necessary to make a building capable to withstand the seismic actions defined at points [...] >> (DM LL PP 1986, C.9.1.1), that is, as for new structures; while the latter is << [...] the execution of one or more operations regarding single structural elements of the building to achieve a greater degree of safety, without substantially altering the global behavior [...] >> (DM LL PP 1986, C.9.1.2). Seismic improvement started to be perceived as particularly apt for listed architectural heritage.

A few years later, the Melandri Law (L.490 / 99) defined restoration with the awareness that many historic buildings are in seismic territories: << Restoration aims to maintain the integrity of the object and to ensure the preservation and the protection of its cultural values. When the cultural heritage asset is located in areas declared at seismic risk according to the current codes, restoration has to include the intervention of structural improvement >> (DL 1999, Article 34).

In particular, on the reactions of buildings subjected to earthquakes, Giuffré wrote: "The earthquake does not chaotically disintegrate houses but selects the weakest structural parts and technological solutions" (Giuffré 1993). He also rose awareness on the issue of safety in the approach of conservation, saying << [...] first of all we need to know what has to be preserved, and from this knowledge, to come up with 'how' to preserve it safely. [...] >> (Giuffré 1993).

Further studies on construction techniques and materials characterized the end of the 20th century, leading to the production of manuals for repair interventions, grounded in the specific geographic reality of Italy. For instance, Paolo Marconi that in those years collaborated to write the *Manuale del recupero del Comune di Roma*, (Manual for restoration of the Municipality of Rome), stressed the importance of studying local techniques of construction and focusing on the behavior of the buildings during earthquakes by observing the damage modes (Marconi 1999). Several manuals of those years at this regard could be consulted (Giovanetti 1992; Mannoni 1995; Giuffré and Carocci 1997; Giuffré et al. 1999; Gurrieri 1999; Giovanetti 2000).

Years later, the concepts of improvement and equivalent safety were confirmed with the elaboration of guidelines for cultural heritage in 2006, with an updated version in 2011, still in use to date (MiBAC 2011). Such guidelines are still a reference today and aimed << [...] to become less prescriptive and more in the form of recommendations, linking the technical-scientific profile to the historical-critical analysis [...] >> (Tempesta 2012). They simultaneously focus on the original materials and the reconstruction of the seismic history of the monument, highlight the interventions executed after seismic events and stress the ability to dialogue between different disciplines such as architecture, history, geotechnics, and structural engineering. Such approach is in line with what stated by the Charter of Cracow, 2000: << The role of conservation and restoration techniques is closely linked to interdisciplinary scientific research on specific materials and specific technologies used in construction, repairing and restoration of the built heritage. The chosen operation must respect the original function and ensure compatibility with existing materials, structures, and architectural values. New materials and modern technologies must be rigorously tested, compared and adapted to the real conservation needs [...] >> (ICOMOS 2019).

Important earthquakes, that affected Italy in the 20^{th} century, motivated the close attention devoted in those years to the seismic issues and interventions on historical masonry buildings. Section 1.2 examines in detail the studies carried out specifically on churches that those years have stimulated.

1.2 Vulnerability of churches: first steps and concepts

Churches are a building typology vulnerable to earthquakes. In Italy, since the second half of the 20th century, post-earthquake damage surveys have provided abundant material to investigate their structural response to seismic action. Such damage observations have motivated the development of procedures for the seismic vulnerability assessment addressed specifically to churches. Methodologically, the studies of damage and vulnerability were, indeed, always carried out in parallel.

The damage interpretation after the Friuli earthquake, 1976, was the starting point for all the following research on churches. The severity of this seismic event, which reached *Mw 6.5,* caused extensive damage to all masonry construction. Initially, a methodology was developed for assessing the vulnerability of ordinary residential masonry buildings. Such procedure, however, was unsuited to interpret the behavior of churches and the damage that could possibly develop. Doglioni et al. used the data collected from the Friuli experience to develop the first methodology for the damage and vulnerability assessment of this building category. It consists on considering the independent behavior of each part, or *macro-element*, of the church (e.g. apse, transept, nave, façade) associating each one to recurrent failure *mechanisms* and to its vulnerability. The method distinguished between typical and specific vulnerability. The former concerns the plane-volumetric configuration of the building and is related to macro-elements, while the latter derives from the presence of factors causing locally a state of weakness (Doglioni et al. 1994). The identified failure mechanisms may be grouped according to two distinct types of structural behavior: *in-plane modes*, when the failure is due to shear, and *out-of-plane modes*, when hinge lines are formed and wall elements rotate around them developing a kinematic chain (Giuffré 1991). These two types of behavior appear also in earlier manuals, for instance in Rondelet's manual (Rondelet 1831).

All these concepts and data available for the interpretation of structural behavior stimulated the elaboration of a first procedure with a template, or form, to assess the damage for churches (Doglioni et al. 1994). A first experience of application of such approach was performed with the Modena and Reggio Emilia earthquake in 1987. After the damage observations in the earthquakes of Lunigiana and Garfagnana, 1995, (Angeletti et al. 1997) and of Umbria-Marche, 1997, (Lagomarsino 1998), the Ministry of Cultural Heritage (MiBAC) and the Civil Protection Department officially adopted a revised form for the damage assessment of churches.

In those years, after the experience of the Umbria-Marche earthquake of 1997, the Regional Authority of Marche issued a Code of Practice for giving guidance in the design and execution of seismic improvement of historical buildings. The first version was issued in 2000 and a second one followed in 2007, including the experience from interventions performed after the 1997 earthquake (Doglioni and Mazzotti 2007). This book was intended to provide a reference of practice for practitioners operating in the field, merging recommendations on interventions in accordance with building codes and new research advancements on the topic.

In 2004, Lagomarsino and Podestà (2004a) proposed 18 possible failure mechanisms and changed from 3 to 5 the levels of damage classification, referring to the European Macro-seismic Scale, EMS-98 (Grunthal et al. 1998). A Damage Probability Matrix (DPM) was later proposed. It was associated with the macro-seismic intensities and obtained from a statistical analysis of damage data (Lagomarsino and Podestà 2004b). The damage interpretation of the Molise earthquake, 2002, (Lagomarsino and Podestà 2004c) led to improving the number of failure mechanisms from 18 to 28. This change determined a more accurate assessment of vulnerability and damage for churches. A survey form for both damage and vulnerability of churches, classified as *form of level II*, was presented (Podestà et al. 2005, Lagomarsino et al. 2004). This form pays specific attention to construction details that influence the structural response of the building. In the modality of evaluation of vulnerability, for each possible mechanism there are two crucial sections for assessing the vulnerability: one devoted to the presence of seismic protection devices and one to the presence of factors increasing vulnerability (or vulnerability indicators). For the damage assessment section, two types of information are specified: one reporting the pre-existing damage and another on damage occurred during the investigated event (Figure 1- 1). At the end of the assessment, a vulnerability index (i_v) and damage index (i_d) are obtained.

19 – MECCANISMI NEGLI ELEMENTI DI COPERTURA - PARETI LATERALI DELL'AULA				
Presenza del macroelemento in relazione al meccanismo: Si ☐ No ☐				Punta di danno massimo (da 0 a 5): __
Vulnerabilità	Si	No	*Presidi antisismici*	
	☐	☐	Presenza di cordoli leggeri (metallici reticolari, muratura armata, c.a. sottili)	☐ ☐ ☐
	☐	☐	Presenza di collegamento puntuale delle travi alla muratura	☐ ☐ ☐
	☐	☐	Presenza di controventi di falda (tavolato incrociato o tiranti metallici)	☐ ☐ ☐
	☐	☐	Presenza di buone connessioni tra gli elementi di orditura della copertura	☐ ☐ ☐
	☐	☐	……………………………………………………………… .	☐ ☐ ☐
	Si	No	*Indicatori di vulnerabilità*	
	☐	☐	Presenza di copertura staticamente spingente	☐ ☐ ☐
	☐	☐	Presenza di cordoli rigidi, copertura pesante	☐ ☐ ☐
	☐	☐	……………………………………………………………… .	☐ ☐ ☐
Danno	attuale		Lesioni vicine alle teste delle travi lignee, scorrimento delle stesse – Sconnessioni tra i cordoli e muratura – Movimenti significativi del manto – Sconnessioni e movimenti tra gli elementi di orditura principale	☐ ☐ ☐ ☐ ☐
	vecchio		Lesioni vicine alle teste delle travi lignee, scorrimento delle stesse – Sconnessioni tra i cordoli e muratura – Movimenti significativi del manto – Sconnessioni e movimenti tra gli elementi di orditura principale	☐ ☐ ☐ ☐ ☐

Figure 1- 1 Example of a mechanism extracted from the damage form for churches of II level (Podestà et al. 2005).

These fruitful years of researches led to the formalization of the current version for the damage assessment form, approved in 2006 (PCM-DPC-MiBAC 2006). After an earthquake, survey teams, composed of both structural and cultural heritage experts, fill the damage survey form to provide an overview on the damage in the affected territory. Surveyors evaluate the damage for each of the 28 mechanisms according to a level of severity from 0 (absence) to 5 (collapse).

In 2011, the Ministry of Cultural Heritage, in its Guidelines for the seismic safety of cultural heritage (MiBAC 2011), re-proposed the vulnerability assessment approach of Lagomarsino et al. (2004) and Podestà et al. (2005) indicating a synthetic procedure which again checks the presence or absence of seismic protection devices and vulnerability factors for each specific mechanism. Each outcome, conveniently weighted, contributes to obtain an index of vulnerability (i_v) for the whole church building. This is an effective and powerful method. However, it has some limitations, such as the difficulty for the surveyor to assign the partial numerical coefficients to get to a global numerical index (Chesi et al.2013, De Matteis et al. 2017).

In 2017, an automatic form for damage and vulnerability assessment, called "MaChro Form" (Masonry Churches Reconnaissance Operational Form), was proposed (De Matteis et al. 2017). This form originates from the seismic damage and vulnerability form for churches of level II (Podestà et al. 2005) and aims at reducing uncertainties and subjectivity in assigning values to the partial numerical coefficients and compute the global index in assessing the effectiveness of seismic protection devices. The MaChro

Form is thought also for compilers with little experience in survey campaigns. The surveyor is asked to describe the cultural asset through basic information, which is, then, automatically elaborated by a software system that delivers directly the estimated seismic vulnerability (De Matteis et al. 2017). Such automatic tool assigns values extracted from a predefined set that the surveyor cannot modify. If, on the one hand, such tool tries to overcome the subjectivity of judgement in assigning partial coefficients, on the other hand the philosophy of the approach for the assessment remains the same. The long Italian seismic history with its relevant experience of damage and vulnerability assessment for churches constitutes an example for different, foreign territories. Some attempts were done in order to export the methodology, discussed here, to different contexts in the occasion of post-earthquake damage surveys for churches. Marotta et al. (2017b) declares the absence or limited presence of some macro-elements performing the inventory of New Zealander churches affected by the 2011 Canterbury earthquake (Marotta et al. 2015). The applications in different contexts of the Italian methodology have shown its low flexibility and adaptability to different construction cultures. Such connotations have caused difficulties in damage assessment operations. To this regard, after the 2011 Canterbury earthquake in New Zealand a new damage form, called Church Assessment Form—Damage (CAF-D), was proposed (Lagomarsino et al. 2019). It kept the same philosophy of the Italian methodology that associates macro-elements and specific mechanisms. The way the CAF-D is thought suggests a more adaptable layout in order to overcome the low flexibility by avoiding imposing a pre-fixed number of mechanisms. Finally, the damage information collected with the CAF-D made the calibration of vulnerability curves possible for New Zealand churches, in view of a vulnerability form (CAF-V) not yet defined. Figure 1- 2 summarizes the principal events quoted in this section.

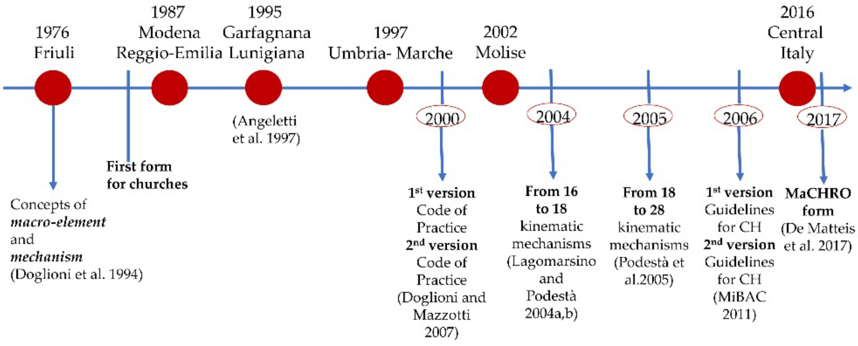


Figure 1- 2 Timeline of the principal events quoted in the section.

CHAPTER

2

THE RESEARCH METHODOLOGY

This section is devoted to explaining the research path and is intended to be a guide in the reading of the thesis.

The present study on the seismic vulnerability for churches has required a multidisciplinary approach. Distinct actions, such as onsite damage and vulnerability assessment surveys, consultations of archives and involvement of local practitioners, administrations, offices of the territories, modeling and interpretation of results, and overlaying of information in a GIS workspace were carried out. The data acquisition process was long and if on the one hand it could appear still not exhaustive, on the other hand it has provided useful indications for the comparison of specificities from one territory to another. Several churches from different geographical areas - Central Italy, Eastern Lombardy, Sicily, and Québec Province in Canada- were considered for the study. The level of detail for each study case differs from one another depending on the specific goal and, at times, on circumstances.

In the Italian cases, the order of the treated areas is a chronological order considering the most recent earthquake that has affected the area.

At the beginning of this research, the 2016 Central Italy earthquake occurred on August 24, followed by other important events on October 26, October 30, and January 18. Such events provided the opportunity to perform post-earthquake damage surveys on churches. Some surveys were carried out in Marche Region in Spring 2017. Some of the visited churches had already been inspected in October 2016. This double-check of the occurred damage allowed observing the damage evolution for consecutive seismic shocks and, at the same time, expressing some considerations on the performance of structural improvement interventions performed in the past years (Parisi et al. 2018).

The consultation of the damage documentation of the churches affected by the 2012 Pianura Padana Emiliana earthquake, stored in the seismic archive at the Superintendence of Brescia, was fundamental for understanding the influence of the specificities of this territory in the structural response.

The other church case treated for Italy is located in Eastern Sicily. Even if just one case study, it was added because it shows different characteristics from the churches treated in the other two areas. It shows a church-fortress with a plan configuration that is exemplificative for other churches of this area of Sicily.

In order to arrive to a seismic vulnerability assessment procedure, the research includes the following steps:

- Step 1. The identification of the territories

  Two geographical areas, Central Italy and Eastern Lombardy, and a specific case in Eastern Sicily, are chosen from the Italian territory as exemplifications of materials, techniques of construction, seismic history, and construction and history culture that strictly influence the local seismic awareness. This is, in turn, reflected in the approach to seismic damage prevention. These territories have different levels of seismicity and different history that have consequently characterized the construction culture. All these aspects are considered in this dissertation as elements that inform the local construction culture. The availability of construction materials close to the building site simplifies the construction process and affects the development of techniques proper to that territory; consequently, similar church typologies show different materials and techniques of construction that influence the response of the structure (Tateo and Sferrazza Papa 2019).

- Step 2. The identification of case studies

  The case studies were selected within the above-mentioned geographical areas for different reasons, i.e. the accessibility of data, similarities, knowledge on the structural interventions, and characteristic features common in those territories.

  In the case of Central Italy and Lombardy, the damage documentation makes it possible to interpret the seismic response in connection with the materials and construction techniques, to formulate hypotheses and, in some cases, to elaborate numerical global or local models to study the response. In this process, there should be also an increase of the knowledge of construction tradition in relation to seismic damage. The information derived from observation, interpretation during onsite visits or archive consultations, together with modeling and analysis results provides a wider comprehension of the occurred damage; at the same time, it emphasizes what is the important information to be observed and considered during a seismic vulnerability assessment of a church.

The case of church-fortress investigated allows to observe the structural response of this type of churches that can be found in Sicily that is the result of its construction and historical culture.

- Step 3. Formulation of questions

  The specific aspects related to the territory, or territorial specificities, pointed out in steps 1 and 2 above, raised important questions such as:

  - What structural characteristics, material properties or geometrical dimensions and proportions increase the vulnerability?
  - What are the territorial specificities that should be observed?
  - What is important to do in a preventive phase analysis?
  - How can these steps be implemented in a flexible seismic vulnerability assessment procedure which considers these territorial specificities?

Figure 2- 1 summarizes the steps of the research methodology.

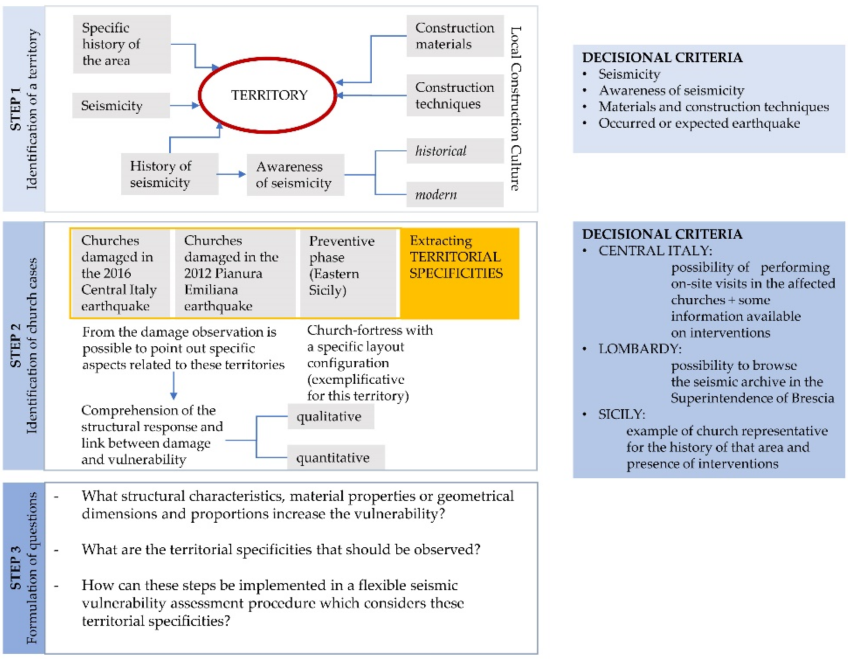


Figure 2- 1 Summary of the research steps.

CHAPTER

3

MACERATA PROVINCE: THE 2016 CENTRAL ITALY EARTHQUAKE

This section is devoted to Central Italy, the seismicity of which is well known; in this regard, the 1997 Umbria-Marche and the 2016 Central Italy earthquakes are the most recent major events. In this region historical buildings suffered, indeed, many earthquakes, and, for this reason, they often underwent repair and retrofitting interventions over their lifetime. For a group of churches inspected after the 2016 Central Italy earthquake the state of damage detected in the survey is described and discussed. One church from this sample has been chosen for developing a case study, being representative of an architectural style common in the area; this church was subjected to interventions in different periods. A comparison of the damage pattern after the two major earthquakes mentioned above has been performed. Finally, some considerations on the territorial specificities are expressed.

The geographical area treated in this chapter is the province of Macerata in Central Italy (see in Appendix Ia the map of Italy). It is highly prone to earthquakes, experiencing in the last decades strong seismic events that have seriously affected the cultural heritage building stock. The frequent occurrence of earthquakes keeps high the level of seismic awareness, both historical and modern (as defined in the Introduction), that is reflected in the construction and intervention practice. Important earthquakes occurred in the last decades in this area: the 1997 Umbria-Marche and the 2016 Central Italy earthquakes, both damaging many churches, and L'Aquila earthquake of 2009, more on the south. A seismic event usually constitutes a test for the interventions performed in the previous period. The 1997 Umbria-Marche and the 2002 Molise earthquakes have pointed out the effectiveness of the strengthening and repairing interventions, executed in the years preceding the events (Tobriner et al. 1997; Lagomarsino 1998). As observed in (Lagomarsino and Podestà 2004a; Binda and Saisi 2005; Calderini 2008; Modena and Binda 2009), modifications and structural interventions could increase the vulnerability of the building. Over the years, the practice of interventions on historical buildings has been influenced by National Building Codes and Charters of Restoration (ICOMOS). Many churches that had undergone massive interventions, that were incompatible with the weak state of the building, reported severe damage. After the 1997 Umbria-Marche earthquake, a specific sensibility on the intervention practice developed. In 2000, the Regional Authority of Marche published some recommendations in support of practitioners, with indications for reconstruction and repairing interventions (Doglioni and Mazzotti 2007). Several churches, damaged by the 1997 Umbria-Marche earthquake, underwent interventions according to such guidelines. The 2016 Central Italy earthquake has provided the opportunity to observe the influence of the interventions performed (Parisi et al. 2018; Penna et al. 2019). This earthquake covered a large area causing extensive damage on church buildings. The damage surveys have provided abundant documentation useful for elaborating considerations on the vulnerability of the churches in relation to the territorial specificities proper of this area.

This chapter, together with Chapters 4 and 5, contributes to elaborate the proposed territorial approach. Among the visited churches, San Salvatore in Acquapagana (MC) was chosen to study the occurred damage with particular reference to the roof intervention, occurred after the 1997 Umbria-Marche earthquake. Such intervention is common to other churches in the same area. Indeed, after the observed post-earthquake damage in 1997 some recommendations on the intervention practice were elaborated (e.g. Doglioni and Mazzotti 2007). At the end of this chapter, some considerations on the characteristic of the churches in the seismic vulnerability perspective will be summarized.

3.1 THE SEISMICITY OF THE AREA

The churches that are part of the sample treated in this chapter, are all in the province of Macerata. This area was classified as highly subjected to earthquakes already in 2003 (OPCM 3274 2003). The category of seismicity assigned at the time was 1 (high seismicity) and 2 (moderate-high seismicity) depending on the different municipalities. Today, reference values for the expected peak ground accelerations are provided by the INGV (Istituto Nazionale di Geofisica e Vulcanologia), referring to the OPCM 3519 2006 (OPCM 2006). Figure 3- 1 shows the values of peak ground acceleration on rock for the concerned area that shift from the range of 0.175-0.200g on the coast, to 0.150-0.175g for the interior, to 0.225-0.250g close to Umbria, with reference to a return period of 475 years. In particular, the area of Serravalle di Chienti, where the church to be studied is located, shows values of acceleration comprised between 0.225 and 0.250. The historical seismicity of Serravalle di Chienti in the Italian Macro-seismic Database (Locati et al. 2019) as reported in Table 3- 1 testifies to the moderate-high seismicity of the area. Most of the listed events belong to the seismic sequence corresponding to the 1997 Umbria-Marche earthquake, whose epicentral area is labelled as Appennino Umbro-Marchigiano.

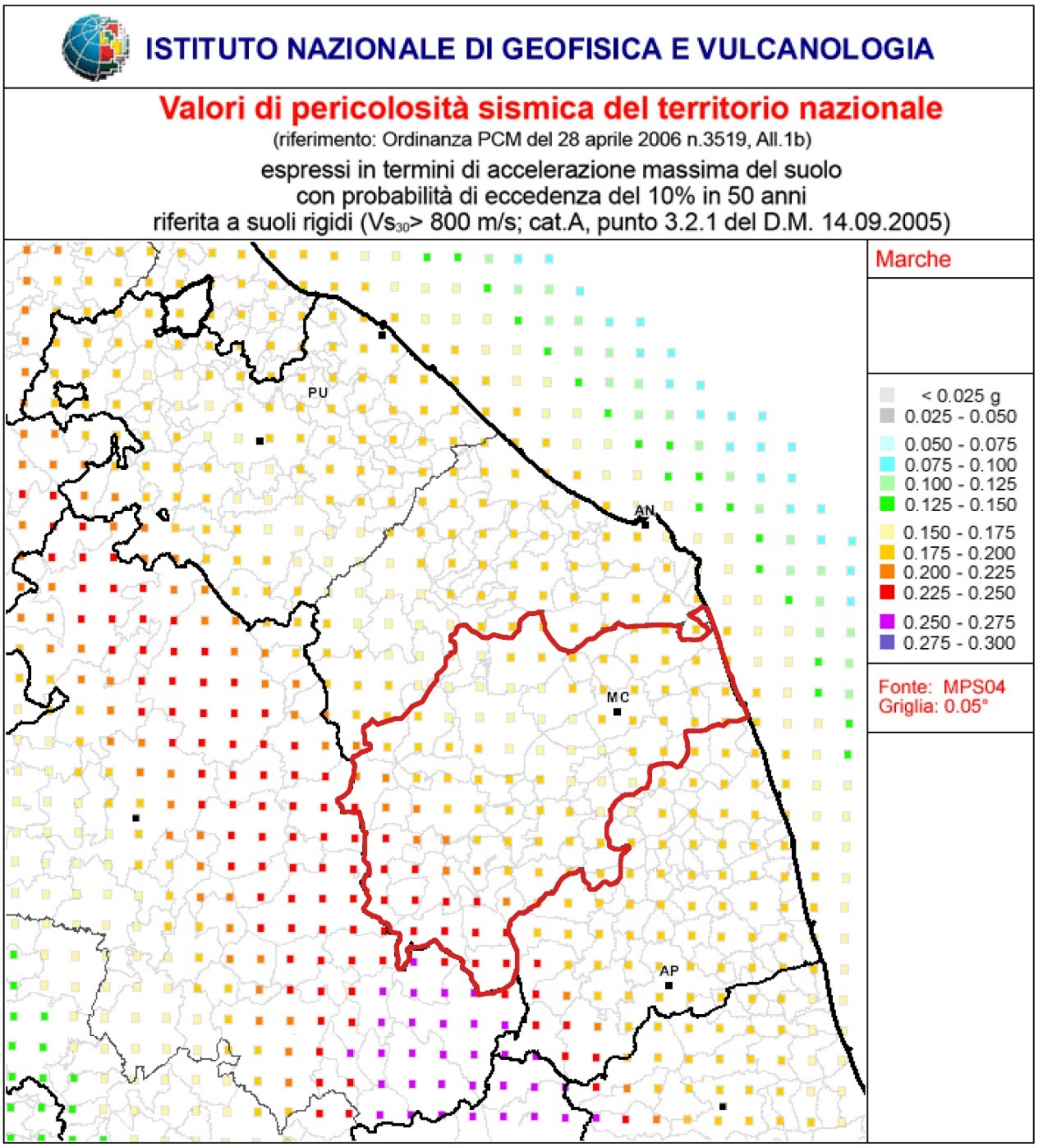


Figure 3- 1 Seismic hazard map for Region Marche in terms of PGA for a probability of exceedance of 10% in 50 years. More specifically the Province of Macerata is highlighted with a red line.

Table 3- 1 The major seismic events for Serravalle di Chienti area. Data source (Locati et al. 2019). Note: The gap between the 1279 and 1785 event is probably due to the lack of recorded data, but earthquakes occurred in the area cocerned.

Microseismic Intensity	Date - Time	Localization	Epicenter Intensity	Moment Magnitude
10	30 04 1279 18:00	Camerino	9	6.31
8	03 05 1785 02:30	Alta valle del Chienti	7	5.14
6	28 07 1799 22:05	Appennino Marchigiano	9	6.13
NF	10 09 1919 16:57:20	Piancastagnaio	7-8	5.32
6-7	19 09 1979 21:35:37	Valnerina	8-9	5.86
4	13 10 1986 05:10:01	Appennino Umbro-Marchigiano	5-6	4.65
3	03 07 1987 10:21:58	Porto San Giorgio		5.09
4-5	04 06 1993 21:36:51	Nocera Umbra	5-6	4.50
4-5	05 06 1993 19:16:17	Gualdo Tadino	6	4.74
4	15 07 1997 08:51	Appennino Umbro-Marchigiano	4-5	3.69
5-6	03 09 1997 22:07:30	Appennino Umbro-Marchigiano	5-6	4.56
5	07 09 1997 23:28:06	Appennino Umbro-Marchigiano	5-6	4.38
4-5	09 09 1997 16:54	Appennino Umbro-Marchigiano	5-6	4.07
4-5	10 09 1997 06:46:51	Appennino Umbro-Marchigiano	5	4.16
7	26 09 1997 00:33:13	Appennino Umbro-Marchigiano		5.70
6-7	03 10 1997 08:55:22	Appennino Umbro-Marchigiano		5.25
6	06 10 1997 23:24:53	Appennino Umbro-Marchigiano		5.46
7	14 10 1997 15:23:11	Appennino Umbro-Marchigiano	7-8	5.65
4-5	09 11 1997 19:07:33	Appennino Umbro-Marchigiano	5-6	4.90
5	07 02 1998 00:59:45	Appennino Umbro-Marchigiano	5-6	4.43
4-5	16 02 1998 13:45:45	Appennino Umbro-Marchigiano	5	4.03
5-6	21 03 1998 16:45:09	Appennino Umbro-Marchigiano	6	5.03
5	26 03 1998 16:26:17	Appennino Umbro-Marchigiano	6	5.29
5-6	05 04 1998 15:52:21	Appennino Umbro-Marchigiano	6	4.81
4-5	11 08 1998 05:22:59	Appennino Umbro-Marchigiano	5-6	4.53
4-5	29 11 1999 03:20:34	Appennino Centrale	5-6	4.38
NF	09 12 2004 02:44:25	Zona Teramo	5-6	4.18
2-3	12 04 2005 00:31:52	Maceratese	4-5	4.16
4-5	15 12 2005 13:28:39	Valle del Topino	5-6	4.66
3-4	10 04 2006 19:03:36	Maceratese	5	4.51

3.2 The 2016 Central Italy Earthquake: the Major Shocks

Between August 2016 and January 2017, Central Italy was struck by a series of strong shocks which put under serious risk large part of the cultural heritage assets, as declared by the activity report of the Regional Seismic emergency unit of the 2016 Central Italy earthquake (Unità di crisi MiBACT 2017). Figure 3- 2 synthesized the seismic swarm that occurred in the area. The epicenters of the series of shocks migrated from south to north, moving south again with the shocks of January 2017.

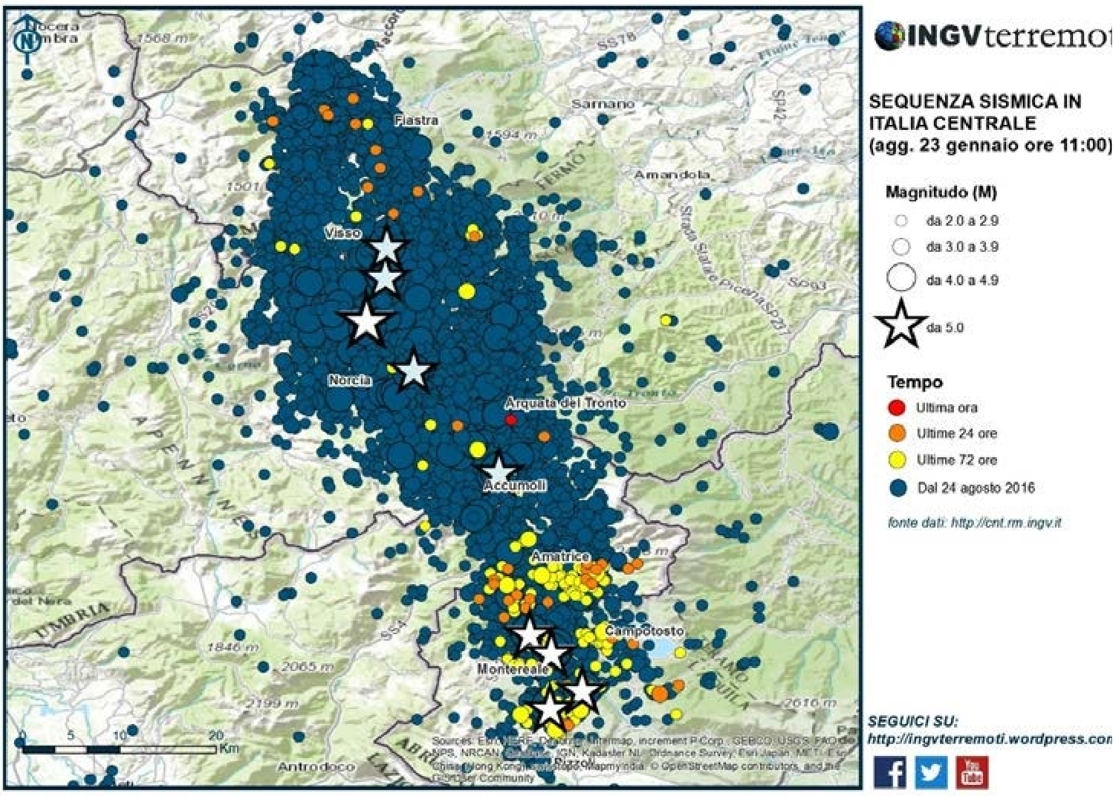


Figure 3- 2 Seismic swarms of Central Italy. The white stars represent the events with a magnitude greater than 5. The first shock, Mw 6.0, was felt in Accumoli on August 24; shocks in the area of Castel Sant'Angelo with 5.9 Mw on October 26, and in Norcia, 6.5 Mw, October, 30; the fourth major event occurred on January 18, 2017 in the area of Montereale – Capitignano, 5.5 Mw (INGV 2017).

The INGV map shows the substantial portion of the territory that was impacted in those months, from August to January. The seismic swarm of the 2016 Central Italy earthquake covered a large area of approximately 30 000 km^2, interesting about thirty different dioceses (Penna et al. 2019). In this period, four major shocks hit the area. The first seismic event occurred on August 24 (6.0 *Mw*) with epicenter in Accumoli, the two strongest ones for the area considered here arrived on October 26 (5.9 *Mw*) with epicenter close to Visso and on October 30 (6.5 *Mw*) with epicenter in Norcia, and a further one, in terms of time and epicenter distance, occurred on January 18 (5.5 *Mw*) with epicenter in Montereale – Capitignano. They were generated between the two seismogenic units Colfiorito-Campotosto and Vettore-Porche-Bove. Figure 3- 3 shows the 10 visited churches together with the two seismogenic units where the epicenters are located.

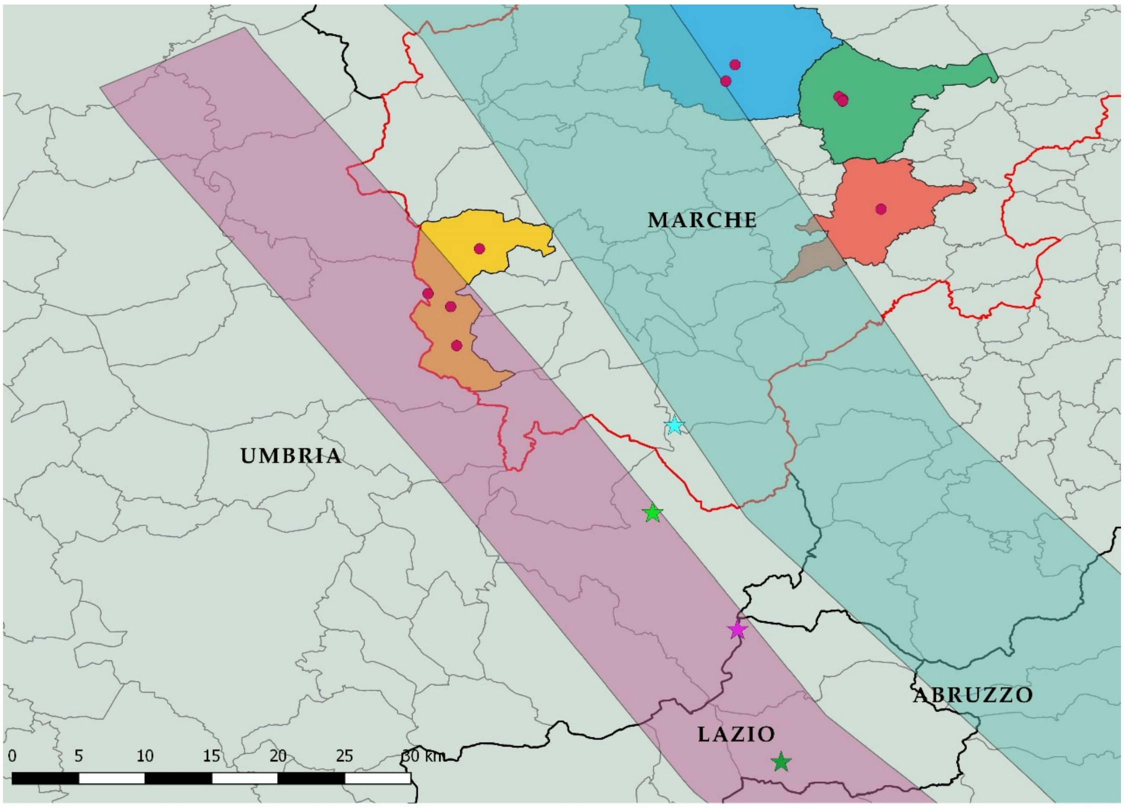


Figure 3- 3 The set of selected churches in the territory of Macerata (red line) in the municipality of Serravalle di Chienti (yellow area), San Ginesio (red area), Tolentino (green area), and San Severino Marche (blue area). The image also shows the two seismogenic units Colfiorito-Campotosto (purple area) and Vettore-Porche-Bove (light blue area) (DISS Working Group, 2018).

3.2.1 The sample of the damaged churches

The visited churches are located in different municipalities of the province of Macerata, as shown by the colors in Figure 3- 3. This aspect explains the differences in terms of construction material, changing from brick in the hinterland of Macerata, to stone masonry, towards the Umbria border, as well as the associated construction techniques. Figure 3- 4 shows the churches for which damage was inspected; some information is collected in Appendix Ib. Within this sample, it is difficult to identify a single typology, but some considerations may be made based on the occurred damage.


(a) **(b)**


(c) **(d)**


(e) **(f)**


(g) **(h)**

(i)

(l)

Figure 3- 4 The churches visited after the four seismic shocks: **a**. San Lorenzo in Dignano (Serravalle di Chienti); **b.** San Salvatore in Acquapagana in Serravalle di Chienti; **c.** Santa Maria di Plestia (Serravalle di Chienti); **d.** Santa Lucia in Serravalle di Chienti; **e.** Duomo Vecchio in San Severino Marche; **f.** Santa Maria del Glorioso in San Severino Marche; **g.** San Francesco in Tolentino; **h**. Santa Maria Nuova, La Tempesta in Tolentino; **i.** SS. Crocifisso in Tolentino; **l.** Collegiata di Sant'Andrea in San Ginesio.

The façades show a simple configuration, reproducing the shape of the roof could be called *gable-façade*. The Collegiata di Sant'Andrea, for such aspect, constitutes an exception, showing a rectangular soaring gable, called 'facciata all'Abruzzese', well-known for its high vulnerability. A characteristic of the hut-shape churches that influences their vulnerability is in the openings (round rose windows or other shapes) in the upper part of the façade, easily triggering the activation of an in-plane façade mechanism (mechanism 3 according to the reference abacus) and the overtopping of the gable (mechanism 2). When the façade is tall, with height greater than width, and the construction material is brick masonry, a pair of pilasters (e.g. Figure 3- 4g, Figure 3- 4h) is usually present. These are made of the same material, contributing to increase the stiffness of the façade wall in the portion where they are built.

In just 2 cases over 10 the bell tower is annexed to the façade; in other cases, it tends to be built in the back side of the church in proximity of the triumphal arch of the apse area. The belfry is generally made of a different material or, otherwise, of the same material, but with a different pattern compared to the rest of the bell tower. The hypothesis elaborated to this regard is that, due to the seismicity of the area, the belfry may have been damaged and repaired at some time during the life of the church.

Another peculiar characteristic of the churches in this territory is the use of brick for the vaults. They were found both with the bricks laid vertically (soldiers) (Figure 3- 5a) and on the flat side (stretchers of particularly small thickness). The latter, thin brick vaults (*volte in folio)* (Figure 3- 5b) were common in the 18th century, when they were often inserted to hide the trusses of the roof in the naves. Such practice of construction really added an element of vulnerability. Differently from the case of many churches from Lombardy, no stiffening ribs were found. These thin brick shells were easy, fast and relatively economic to be realized. This is a common practice of the Mediterranean area that has been adapted to the needs of the particular territory where it was in use. Some additional information can be consulted in Appendix Ic. Another system of vault covering is realized in the area with reeds, the so-called *camorcanna* (reed-vault) (Figure 3- 5c, Figure 3- 5d).

(a)

(b)

(c)

(d)

Figure 3- 5 Types of vaults found in the visited churches: **a.** vault made of bricks laid vertically; **b.** vault realized with the bricks laid horizontally; **c**. reed vault with the detail of the connection with the walls; **d**. reed vault with the detail of the supporting timber structure.

3.2.2 THE OCCURRED DAMAGE

Figure 3- 14 reports the damage, distributed by macro-elements, that was surveyed with the damage forms for churches (DC PCM-DPC MiBAC 2006), after the four major shocks. The histogram shows that the most damaged macro-elements are the façade, the vaults, the triumphal arches, and the bell tower. This is in agreement with the specificities exposed in Section 3.2.1.

Some specifications on the occurred damage are reported in the following:

- The *façade* suffered both in-plane and out-of-plane mechanisms. The former is mainly due to the presence of openings, still existing or closed in the past by infilling; the latter was observed only in few cases in the visited churches, 4 out of 10;

- The *gable* suffered damage level 3 for 6 out of 10 churches. In the specific case of the *Collegiata di Sant'Andrea* (Figure 3- 6a), a rotation of 8 cm of the upper rectangular portion of the gable occurred, due to the overhanging characteristic of this element (Figure 3- 6b).


(a) **(b)**

Figure 3- 6 gable of Collegiata di Sant'Andrea in San Ginesio: **a.** the church; **b**. the detail of the gable (Parisi et al. 2018)

- *Walls.* When the construction material is brick masonry, the cracks are usually stair stepped cracks, contrary to what use to occur in stone masonry walls where the damage is characterized by inclined and irregular cracks (Figure 3- 7).

(a) **(b)**

Figure 3- 7 Example of cracks in the masonry walls: **a.** brick wall; **b.** stone wall.

- *Vaults.* This structural and architectural element suffered damage both in the case of thin brick shells and in the case of 'voussoir' vault (see previous Figure 3- 5).
- *Colonnade.* When the church has more than one nave, the colonnade responds to the longitudinal loads; such response usually generates shear cracks at the level of the entablature when this element exists between the column and the vault level(Figure 3- 8).

(a) **(b)**

Figure 3- 8 Shear cracks in the entablature as longitudinal response of the colonnade: **a.** crack in the entablature of the colonnade; **b.** detail of the shear crack.

- *Roof.* In some cases, the pounding effect of the beams of the roof was ascertained, as in the case of San Salvatore church (Section 3.3.1). In other cases, as in the church of SS Crocifisso, some cracks appeared, following a path through the mortar joints in the longitudinal walls of the nave that support the trusses of the roof (Figure 3- 9).

Figure 3- 9 Trusses of the roof entering the longitudinal wall of the nave. Cracks that start from the portion of the wall where the roof beams are supported.

- *Dome.* Some shear cracks are found in the churches that present this architectural element, in particular in proximity of the openings (Figure 3- 10).

(a) **(b)**

Figure 3- 10 Shear cracks in the domes: **a.** Santa Maria Nuova in Tolentino; **b.** Santa Maria del Glorioso in San Severino Marche.

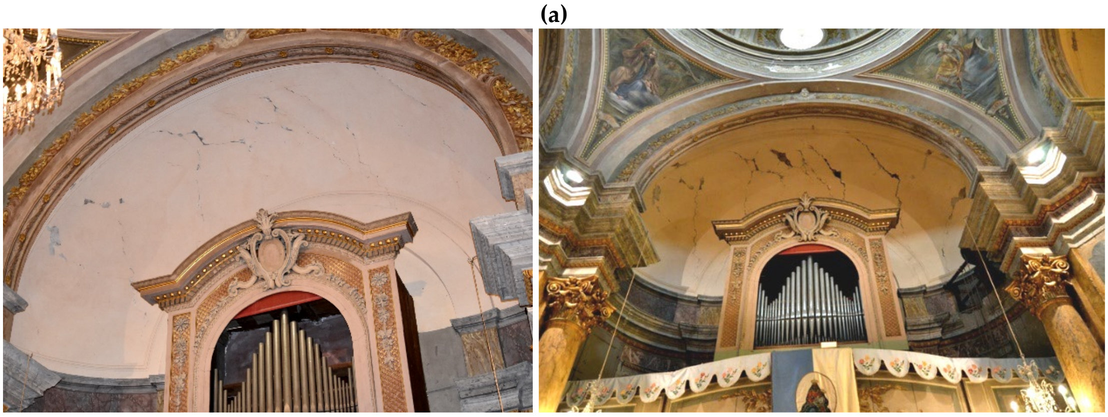
- *Bell towers.* The most common damage reported for this macro-element is due to torsional effects of the upper part (belfry), that depending on the slenderness of the pillars was more or less evident (Figure 3- 11).

(a) **(b)**

Figure 3- 11 Bell tower torsion of the upper part of the belfry of the bell tower of San Lorenzo in Dignano.

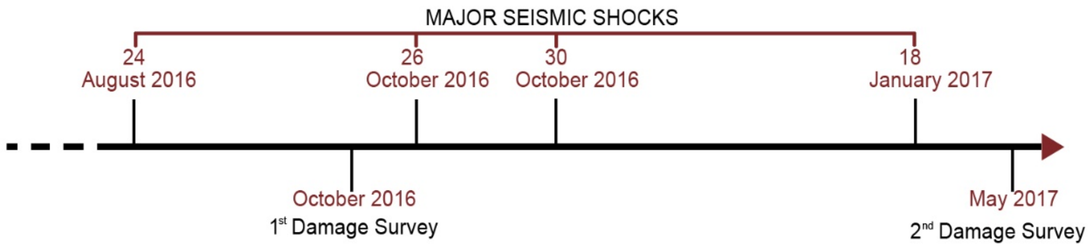
Moreover, some of the churches were visited twice, once after the first event of August 2016, but before the major shocks of October 26 and October 30, and once in spring 2017. This double survey gave the possibility to observe the evolution of damage in some of the churches of the sample. This was the case of San Francesco, Santa Maria Nuova, and SS Crocifisso in Tolentino (Figure 3- 12).

(a)



(b)

(c)



(d)

Figure 3- 12 Evolution of the damage in the churches visited twice: on the left before the shocks occurred on October and on the right after the four shocks: **a.** damage on the interior side of the façade at the second order of San Francesco church; **b.** damage on one of the vaults which cover the niches of Santa Maria Nuova church; **c.** damage on the vault of the nave of SS Crocifisso church; **d.** timeline which clarifies when the surveys were performed together with the major seismic events (Parisi et al. 2018).

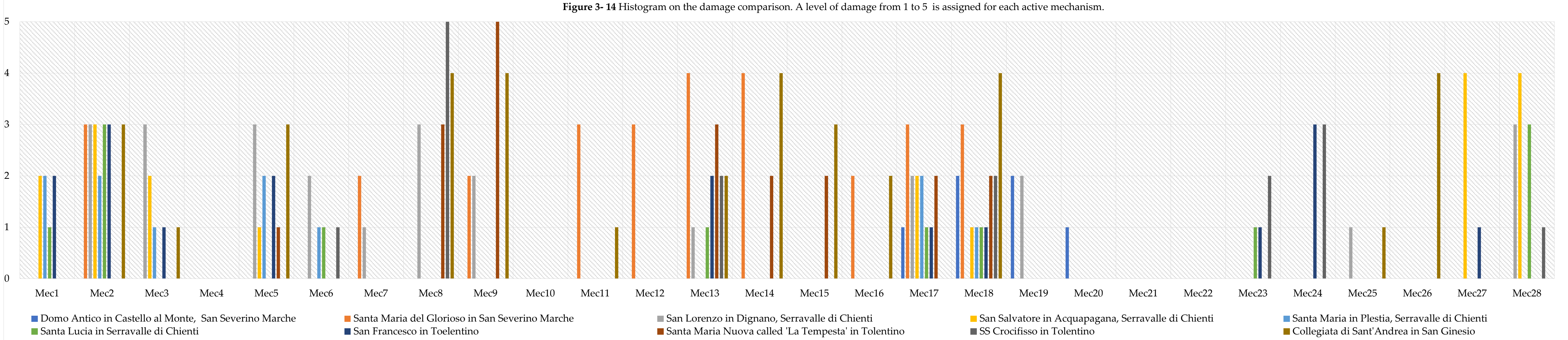
Another condition that it is worth noting is the influence of seismic resistant protection details documented during the damage surveys. This is the case of the tie rods of San Francesco and the steel plate hoop in the drum of Santa Maria Nuova, called La Tempesta. in Tolentino.

In the first case, the tie rods that had been positioned at the level of the impost of the arches and vault provided an effective contribution in stiffening the system and retaining the walls and vault. This is demonstrated by the rupture of the central tie that occurred in August 2016 and by the subsequent ruptures of two more ties with the two main shocks that arrived in October (Figure 3- 13).Immediately they were integrated with an emergency action.


Figure 3- 13 Broken tie rod in August 2016 (Parisi et al. 2018)

In the second case, the presence of a steel hoop, confining the drum, seemed to have influenced the damage. This reinforcement was intended to protect the top elements, but the damage occurred in the inferior vaults, which were more difficult to protect with limited interventions. Cracks were located in the four semi-circular vaults holding the drum and the dome. No serious damage was reported for these last two elements for the entire earthquake sequence. On the contrary, damage to the vaults increased with the following events, bringing one of them to the collapse limit (Figure 3- 12b). Further studies are required to better explain what occurred.

Figure 3- 14 Histogram on the damage comparison. A level of damage from 1 to 5 is assigned for each active mechanism.

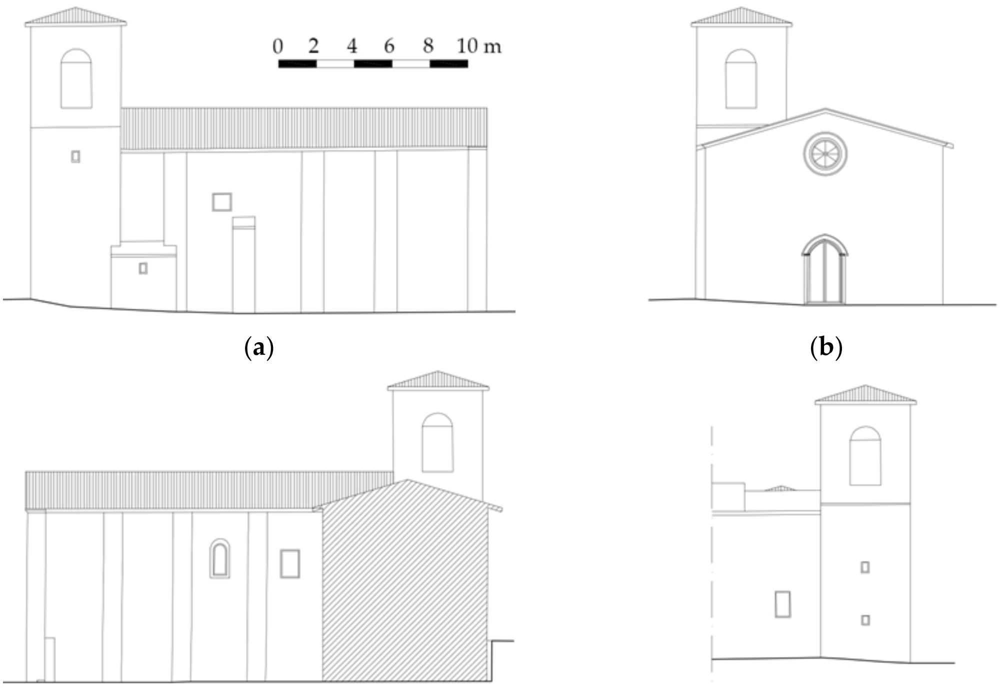


3.3 The case study of San Salvatore in Acquapagana (MC)

St. Salvatore is a medieval church in Aquapagana, a small hamlet in the province of Macerata. This church typology is largely widespread in central Italy: according to (Borri et al. 2019) more than 450 churches of this style have been identified in Umbria by the Italian Conservation and Protection body (Soprintendenza Archeologia, Belle Arti e Paesaggio). The simplicity of the plan configuration of San Salvatore is typical of other church buildings of the area: in (De Matteis et al. 2017) 68 out of 107 damaged surveyed churches, i.e. 64 percent of the total, and in (Carbonari et al. 2017) 350 over 500, 70 percent, are single nave churches. Moreover, in seismic prone areas, the historical buildings are subjected to changes and repairing after the earthquake has produced damage. In this case, the concrete roof of St. Salvatore was replaced after the 1997 Umbria-Marche earthquake with a light timber structure. The implication of such a change is investigated here. For this geographical area, the substitution of the concrete roof with a light timber structure is a territorial specificity, in terms of approach to the intervention practice and seismic awareness.

3.3.1 Description of the church and its features

San Salvatore was built at the beginning of the 11th century. The church is an example of Umbrian-Gothic. The church is part of a monastery, which collapsed in the Umbria-Marche earthquake of 1997 (Sorrentino et al. 2004) and was rebuilt. The smooth stone of the portal and the rose window, the horizontal frame in the middle of the façade, the buttresses and the openings in the lateral walls mark such architectural style (Tassi 2009). The main façade looks west in a simple hut shape, with a round rose window at the second order (Figure 3- 15b). The lateral elevations are characterized by a series of buttresses and few small openings (‘monofore’) in the plane of the walls (Figure 3- 15a, Figure 3- 15c).

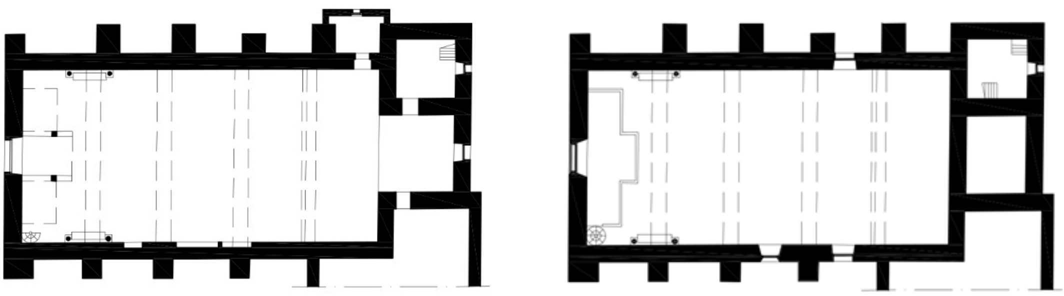


(**a**) (**b**)

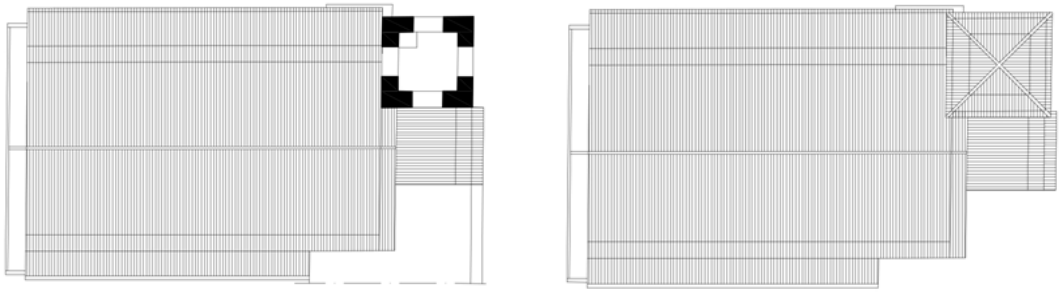
(**c**) (**d**)

Figure 3- 15 Elevations of the church: **a.** north wall; **b.** main façade (west side); **c.** south wall; (d) east wall (Sferrazza Papa and Silva 2018).

The church has a single and symmetric nave with the entrance in the axis of the square apse (Figure 3- 16a) and with a bell tower in the northeast side (Figure 3- 15 d, Figure 3- 16), which according to the archive of Camerino archbishop was moved from its position in the main façade, that it occupied until 1737 (MiBAC-Marche 2017). Inside the church, the arches are the only structural elements that give a rhythm to the straight and long space of the nave. Externally, some of the buttresses, that confine the lateral walls of the nave, are not aligned with the arches of the nave (Figure 3- 16). For this reason, the assumption that they were realized in a second moment was formulated. In the surroundings, a similar church, San Salvatore di Campi, was subjected to such intervention after the 1703 earthquake (Borri et al. 2019).



(**a**) (**b**)

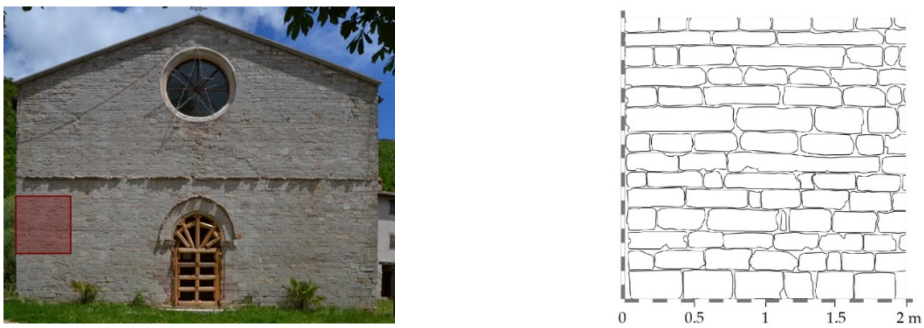


(c) (d)

Figure 3- 16 Plans of the church: **a.** at the level of the entrance; **b.** at the level of the choir; **c.** at the level of the bell tower openings; **d.** at the roof level (Sferrazza Papa and Silva 2018).

The roof has been changed over time, without altering its configuration. In 1960, the wooden roof was replaced with a concrete slab and a concrete ring beam (MiBAC-Marche). This is visible in the photos of the post-1997 earthquake damage recognition (Tassi 2009). Such type of roof was then substituted with a light timber structure (MiBAC-Marche) and some tie rods were added in the arches. These last are visible today and are not present in the photos that document the post-earthquake damage.

The quality of the walls was assessed, based on the procedure recommended in (Borri et al. 2015). Such procedure is in line with the principle of the "rule of the art", that is the totality of all those constructive techniques that guarantee good structural behavior and ensure compactness and monolithism, and that avoid phenomena of instability or degradation (Baila et al. 2011). The thickness of the walls is constant both for the façade and the lateral walls, with an average thickness of about 85/90 *cm*. The walls consisted of three leaves with some cross-stone blocks. The façade wall is studied, in the following, with more detail because it suffered damage, of a different entity, in the Umbria-Marche, 1997, and Central Italy, 2016, earthquakes sample of 200 cm of width and 90 cm of thickness was selected and analyzed (Figure 3- 17).



(a) **(b)**

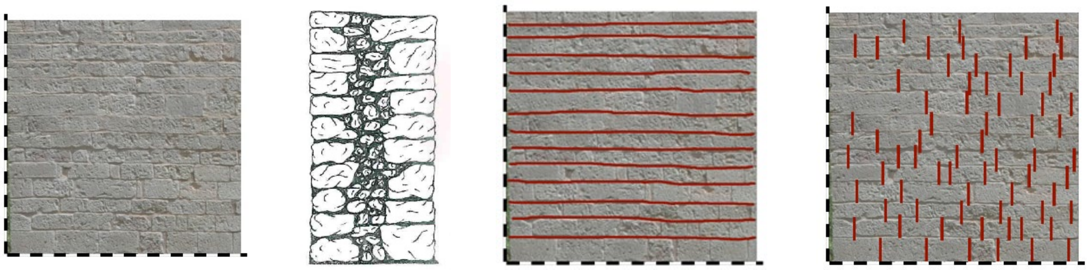
Figure 3- 17 Façade wall sample: **a.** Photo of the façade with highlighted in red the investigated area; **b.** redrawn of a portion of the wall of the façade of the church (2.00×2.00 m) (Sferrazza Papa and Silva 2018).

For the global assessment of masonry quality, the selected specimen is evaluated considering seven factors, which influence the good behavior of masonry. The seven factors are (Borri et al. 2015):

- Good quality of resistant elements "SM", that considers the conservation state and the mechanical properties of bricks or stones;
- Size of the elements, ''SD'';
- Shape of the elements, ''SS'';
- Wall leaf connection ''WC'';
- Horizontal bed joint characteristics ''HC'';
- Vertical joint characteristics ''VC'';
- Mortar mechanical properties ''MM''

The assessment procedure consists of stating for each one of the seven factors that the rule of art is: Fulfilled (F), Partially Fulfilled (PF) or Not Fulfilled (NF). Each parameter is described for every action (vertical, out of plane, in plane) with a score, leading at the end to the definition of the final masonry quality index (IMQ). This index corresponds to different quality categories: A = good behavior of masonry; B = behavior of average quality of the masonry; C = inadequate behavior of masonry.

The stone elements of St. Salvatore church do not exhibit degradation phenomena. The external leaf is made of roughly dressed stones, joined by thin layers of mortar, set up to realize a regular pattern (Figure 3- 18a). The prevalent dimensions of the stone blocks are a length of 30–50 cm, with some exceptions of stone blocks of 70 cm, with a height of 10–25 cm and a thickness of 10–20 cm. The section is supposed made of three leaves with a transversal block between the layers, in accordance with the photo of the 1997 earthquake damage (Figure 3- 18b). The mortar joints are in good condition, well preserved, thin and well proportioned. Horizontal joints characterize all the length of the wall without interruption (Figure 3- 18c), while the staggering of vertical joints is only partially respected because some joints are not in the middle of the stone element (Figure 3- 18d). The global evaluation of the masonry is summarized in Table 3- 2: a section is devoted to show the photographic details of the masonry, another to show the schemes, one more to show the descriptions of the construction techniques and materials and one to show the geometrical stone characteristics; finally, the last section deals with the correspondence of the seven factors with a final evaluation of the quality of the masonry, resulting in being good.



(a) **(b)** **(c)** **(d)**

Figure 3- 18 Analyses of the sample of the façade masonry: **a.** appearance of the façade sample; **b.** three leaf wall with cross blocks; **c.** presence of horizontal joints; **d.** staggering of vertical joints (Sferrazza Papa and Silva 2018).

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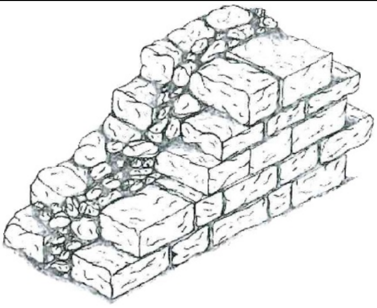
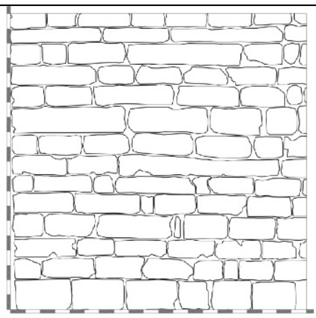
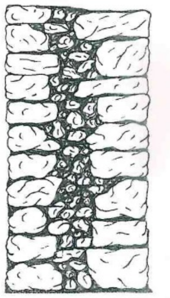
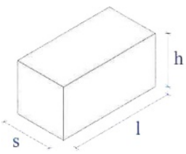
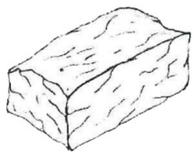
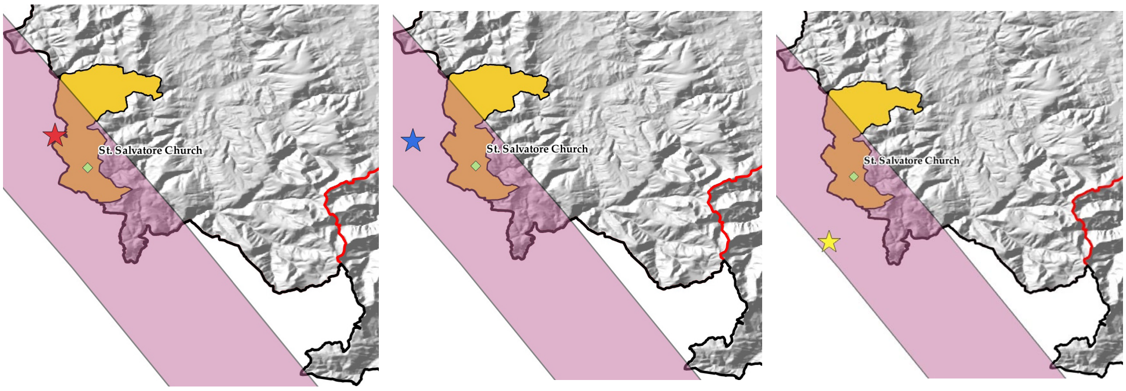
Table 3- 2 Wall quality index evaluation (Sferrazza Papa and Silva 2018).

<table>
<tr><td colspan="11">MASONRY BLOCKS OF SQUARED STONE AND PEBBLES, THREE LEAVES</td></tr>
<tr><td colspan="11">PHOTOS</td></tr>
<tr><td colspan="5"></td><td colspan="3"></td><td colspan="3"></td></tr>
<tr><td colspan="5">Front</td><td colspan="3">Corner</td><td colspan="3">Inner masonry</td></tr>
<tr><td colspan="11">SCHEMES</td></tr>
<tr><td colspan="5"></td><td colspan="3"></td><td colspan="3"></td></tr>
<tr><td colspan="5">Axonometry</td><td colspan="3">Front</td><td colspan="3">Section</td></tr>
<tr><td colspan="11">DESCRIPTION</td></tr>
<tr><td colspan="11">Masonry made up of two leaves built with dressed blocks of stones, with the interposition of a pebble filling. Resistant elements with variable dimensions, but with defined edges. Wall fabric respects horizontal bed joints, partial staggering of vertical joints and presence of transversal stone joints.</td></tr>
<tr><td colspan="11">MATERIAL</td></tr>
<tr><td colspan="11">Spruce travertine squares, also called sponge.</td></tr>
<tr><td colspan="11">GEOMETRY</td></tr>
<tr><td colspan="5">
</td><td colspan="3">Variable elements’ size
(external leaf):
s = 10 ÷ 20 cm
h = 10 ÷ 25 cm
l = 30 ÷ 70 cm</td><td colspan="3"></td></tr>
<tr><td colspan="11">ANALISYS</td></tr>
<tr><td>WL</td><td>MM.</td><td>SS</td><td>VC.</td><td>SM</td><td>HC.</td><td>SD</td><td>Category</td><td>Vertical</td><td>Out of plane</td><td>In plane</td></tr>
<tr><td>F</td><td>F</td><td>F</td><td>PF</td><td>F</td><td>F</td><td>F</td><td>Score method</td><td>A</td><td>A</td><td>A</td></tr>
<tr><td colspan="11">LEGEND</td></tr>
<tr><td colspan="5">SM
SD
SS
WC
HC
VC
MM</td><td colspan="6">Good quality of resistant elements
Size of the elements
Shape of the elements
Wall leaf connection
Horizontal bed joint characteristics
Vertical joint characteristics
Mortar mechanical properties</td></tr>
</table>

3.3.2 The 1997 and the 2016 earthquakes and the church damage

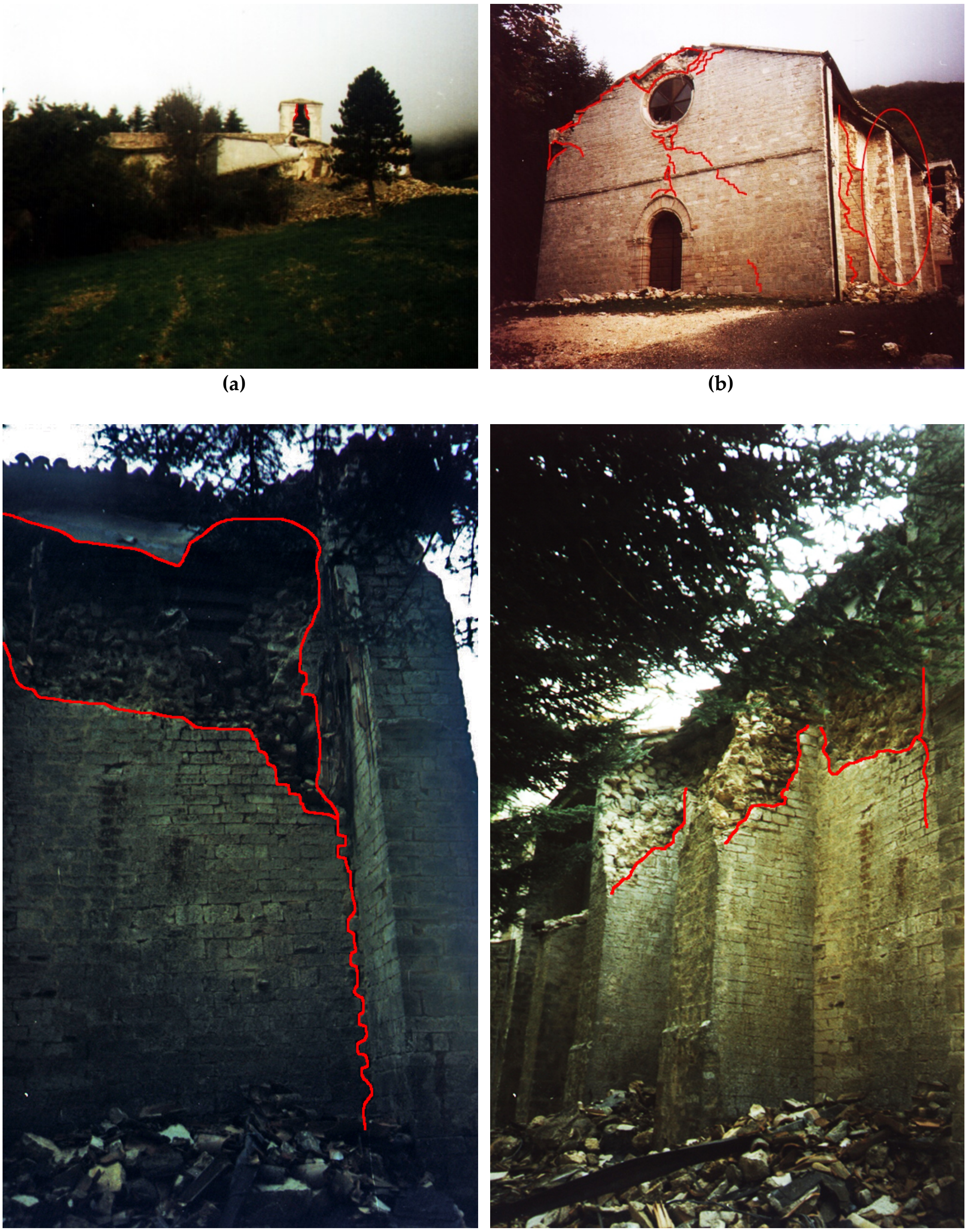
On autumn 1997, the Colfiorito seismogenic unit was the epicentres of the major shocks of the Umbria-Marche earthquake. The first shock was felt (5.70 *Mw*) on September, 26 at 0:33 a.m. It occurred at a depth of 3.51 km, with the epicenter located at 43.022 latitude and 12.891 longitude (Figure 3- 19a). Another shock occurred some hours later, at 9:40 a.m. with 6.01 *Mw* at a depth of 9.87 km. This epicentre was slightly to the north (43.014 lat. and 12.853 long.) (Figure 3- 19b). On October 14, 1997 at 13:23 p.m., a shock of 5.65 *Mw* at a depth of 7.33 km struck Sellano, with the epicenter at 42.898 latitude and 12.898 longitude (Figure 3- 19c).



(a) **(b)** **(c)**

Figure 3- 19 The 1997 major shocks with the location of St. Salvatore church (green square) in Acquapagana and the seismogenic fault system of Colfiorito (purple area). **a.** The red star points out the epicenter of the 26 September 1997 event at 0:33 a.m.; **b.** the blue star specifies the epicenter of the 26 September 1997 event at 9:40 a.m.; **c.** the yellow star indicates the epicenter of the 14 October 1997 event (Sferrazza Papa and Silva 2018).

After this series of shocks, the church was severely damaged. The bell tower lost some parts of the belfry and some shear cracks appeared along the height (Figure 3- 20a). The upper part of the façade in contact with the concrete roof collapsed. Shear cracks appeared in the plane of the façade in the central part between the rose window and the portal (Figure 3- 20b). The lateral wall of the nave showed a separation of leaves with the collapse of a portion of wall, close to the concrete roof (Figure 3- 20c). The buttresses were damaged, losing the upper portion (Figure 3- 20d). In the background of Figure 3- 20b (red oval), some undamaged buttresses exhibit a different texture, especially at the borders of their depth, motivating the hypothesis of a rebuilding phase at some time. It was assumed that the damage followed the path of the later addition.



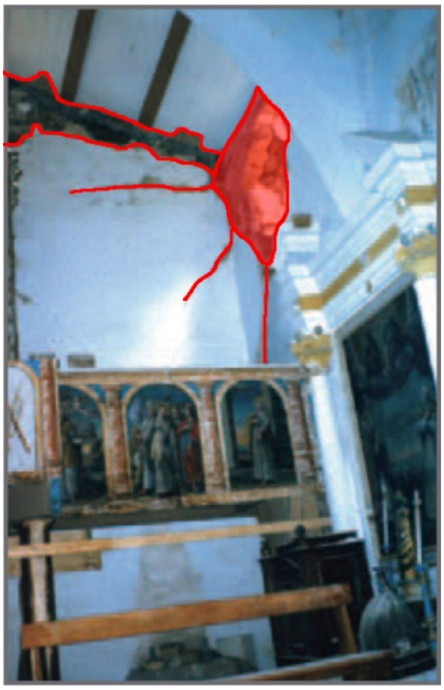
(a) (b)

(c) (d)

Figure 3- 20 The 1997 Umbria-Marche earthquake damage: **a.** collapse of the upper part of the belfry; **c.** collapse of the external leaf of the lateral wall of the nave; **b.** shear crack in the plane of the façade and some collapsed portions of the upper part of the façade and the gable; **d.** collapse of the upper portion of the buttresses (Sferrazza Papa and Silva 2018).

The interior of the church was also damaged. The stiffness of the concrete roof may have caused the shear cracks in the transversal arches (Figure 3- 21a) and the collapsed upper portion of the longitudinal wall is also visible inside (Figure 3- 21b).

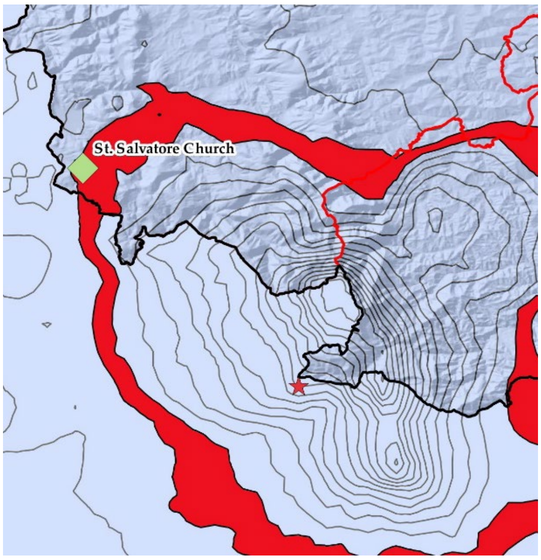
(a)

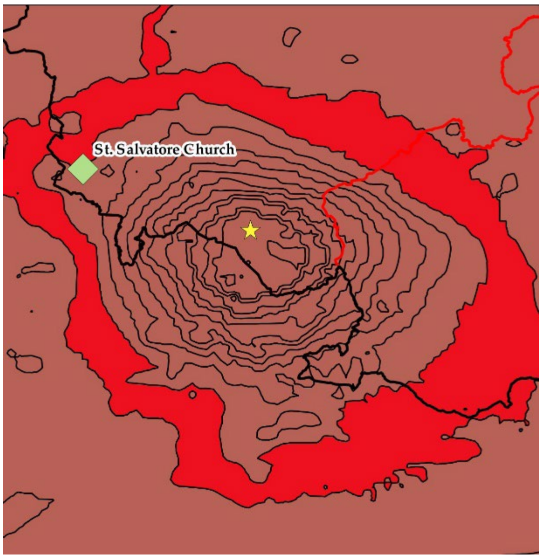


(b)

Figure 3- 21 Post-1997 earthquake damage of the interior of the church: **a.** View of the triumphal arches towards the apse area. Diffuse shear cracks and pounding damage. **b.** Inside view of St. Salvatore church. Partial collapse of the wall and of part of the internal leaf of the wall of the main façade at the tympanum level (Sferrazza Papa and Silva 2018).

Between August 2016 and January 2017, St. Salvatore church was affected by a series of earthquakes and suffered damage. Once more recurring to a GIS software, a superposition of information was possible in order to understand the possible accelerations the church could have undergone. Figure 3- 22 shows the PGA isocurves of the major seismic events of the 2016 Central Italy earthquake, together with the church.





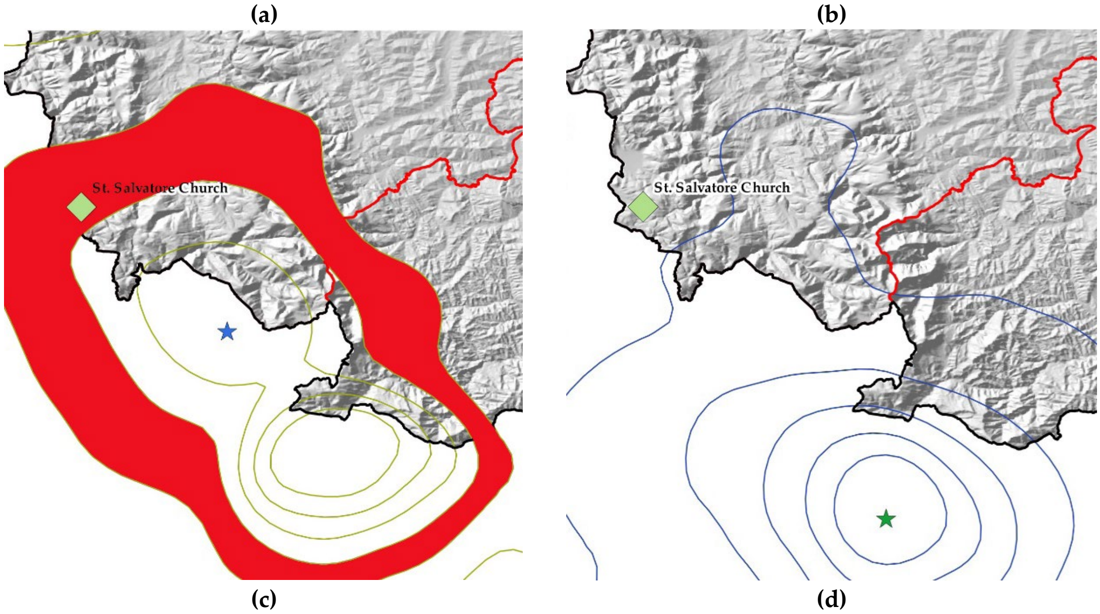


Figure 3- 22 PGA isocurves of the main shocks. **a.** Event of 24 August (12% g); the epicenter is the red star; **b.** event of 26 October (8% g); the epicenter is the yellow star; **c.** event of 30 October (12% g); the epicenter is the blue star; **d.** Event of 18 January (2.5% g); the epicenter is the green star. (Sferrazza Papa and Silva 2018).

The church is in the following PGA isocurves: 12% g on August 24 (Figure 3- 22a); 8% of g on October 26 (Figure 3- 22b); 12% g on October 30 (Figure 3- 22c); 2.5% g on January 18 (Figure 3- 22d).

The damage occurred after this series of seismic events was surveyed and reported in the damage survey form (DC PCM-DPC MIBAC 2006) and here summarized in Figure 3- 23.

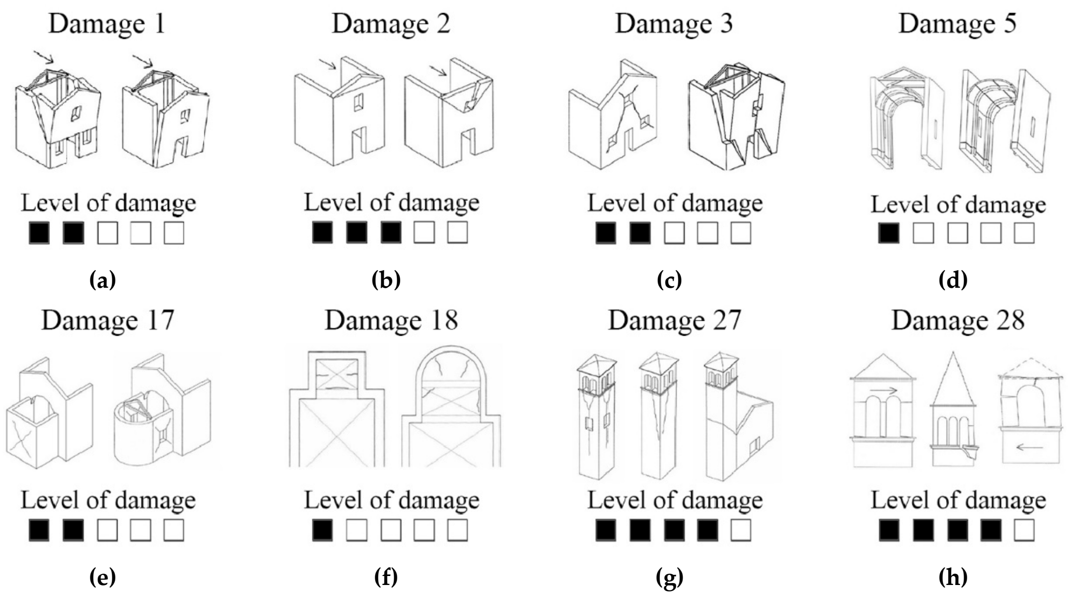
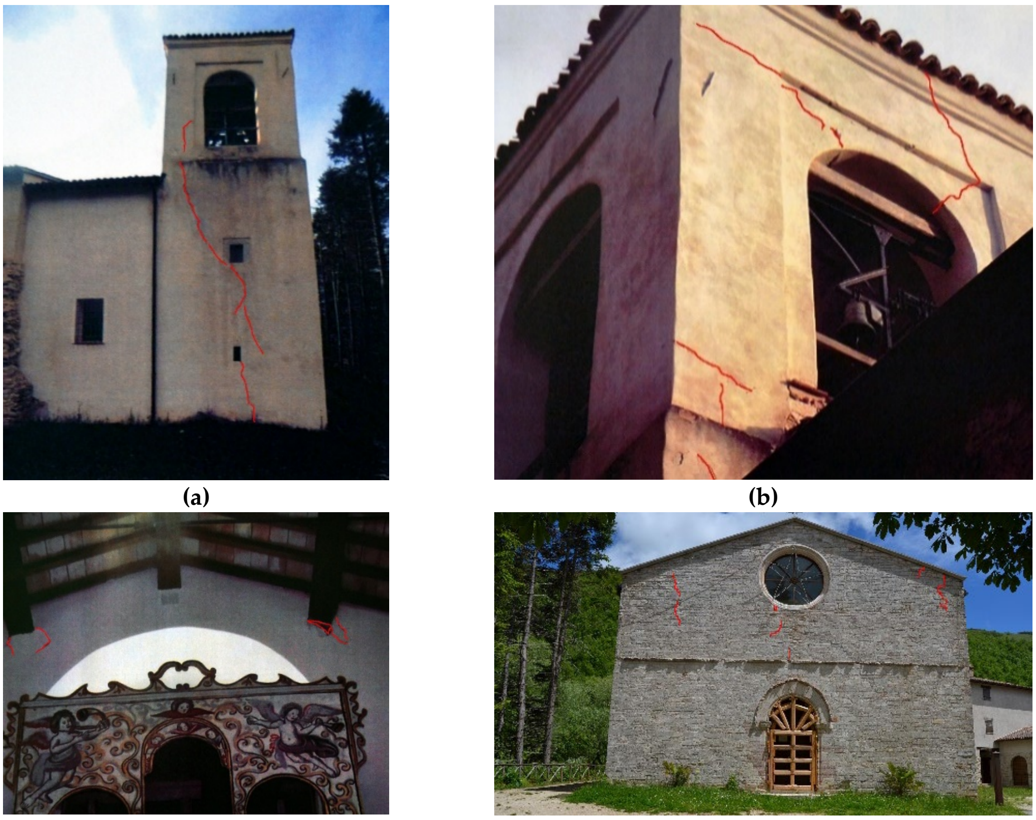


Figure 3- 23 Activated mechanisms and level of damage: **a.** Overturning of the façade; **b.** mechanisms of the upper part of the façade; **c.** in-plane mechanisms of the façade; **d.** longitudinal response of the

lateral wall, in plane; **e.** shear mechanism of the apse; **f.** vaults of the apse; **g.** bell tower; **h.** upper portion of the bell tower (Sferrazza Papa and Silva 2018).

The on-site visit and survey performed in spring 2017 provided the opportunity to collect some photographical documentation that could help explain the damage. The church was less damaged than after the 1997 event. This time, no leaf separating effects were noticed. Some shear cracks were pointed out in the bell tower (Figure 3- 24a,Figure 3- 24b). The presence of the tie rods worked efficiently: indeed, this portion of the tower reacted with a box-like behavior, providing a contrast to the torsional action and causing cracks at the base of the terminal part of the bell tower. Other shear cracks were observed in the façade, probably retracing the discontinuity of the masonry due to the post-1997 earthquake repairing works (Figure 3- 24d). In the upper portion of the façade, in correspondence of the timber beams of the roof a light damage is visible, probably due to their pounding on the façade wall (Figure 3- 24c,Figure 3- 24d). The same occurred inside the church where the timber roof beams enter the arches of the nave (Figure 3- 24e). Finally, a crack at the base of the south wall of the nave was observed (Figure 3- 24f).



(a) **(b)**

(c) **(d)**

(e) (f)

Figure 3- 24 The 2016 Central Italy earthquake damage: **a.** shear cracks on the bell tower along the height; **b.** shear cracks that trace the same path of the damage reported after the 1997 earthquake (Sferrazza Papa and Silva 2018); **c.** Pounding effect of the beam in the façade wall (Sferrazza Papa and Silva 2018); **d.** Crack in the right upper corner in correspondence of the transversal walls of the nave and light damage in the plane of the façade (Sferrazza Papa and Silva 2018); **e.** Cracks in the arches in correspondence of the timber beams of the roof and in the triumphal arch close to the bell tower (Sferrazza Papa et al. 2019); **f.** Crack at the base of the south wall of the nave (Sferrazza Papa et al. 2019).

3.3.3 Numerical analyses

The church was modelled with a structural analysis software (Abaqus 2016). The Finite Element (FE) model used for the analyses had a mesh made of 66055 tetrahedral elements, with a linear formulation (Figure 3- 25).

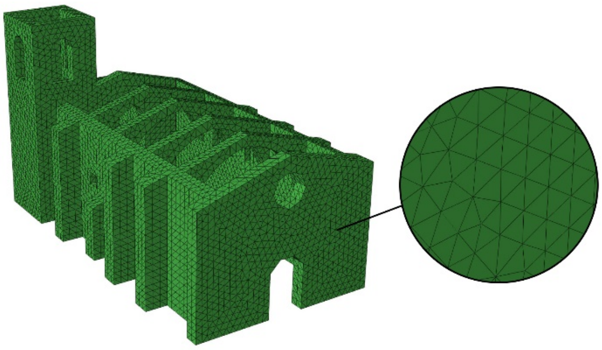
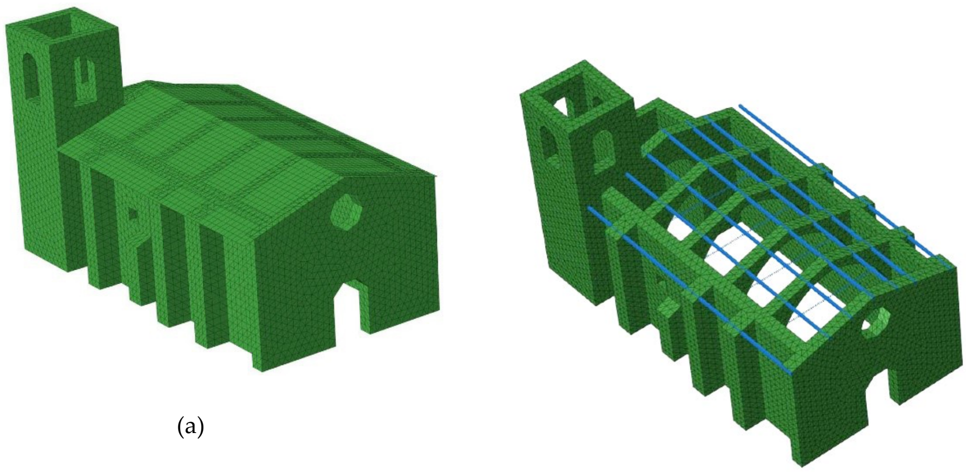


Figure 3- 25 The tetrahedral mesh of the church used for the analyses performed with Abaqus.

In order to investigate the implications of the roof intervention, San Salvatore was modelled simulating the 1997 and the 2016 configurations.
In the first case, the concrete slab was modelled, assumed 5 cm thick, and discretized in 1812 quadrilateral shell elements (Figure 3- 26a). It is connected to the walls and the arches with tie constrains in order to avoid the relative rotation and displacement. A perfect connection between the concrete roof and the masonry structure was assumed in agreement with the photographic documentation of the post-earthquake damage in 1997.
In the second case, the timber beams were modelled as beam elements and the tie rods as linear springs (Figure 3- 26b). The tensile stiffness of the springs (k) was defined as: $k = EA/l = 13000\text{kN/m}$, where E is the Young's modulus equal to 200000 MPa, A is the cross section of the tie rods 30 mm and l, the length of the tie rods, equal to 10.60 m.



(a)

(b)

Figure 3- 26 The two structural configurations of the church: **a.** the church is covered with a concrete roof (1997 configuration); **b.** the church is covered with a light timber roof and has tie rods at the impost of the arches of the nave (2016 configuration).

As observed in section 3.3.1, the masonry walls of St. Salvatore church were made of three leaves. This is a construction technique quite common in the Italian territory. The different composition of the walls in their cross section sometimes compromises the monolithic behavior of the wall. In the performed structural analyses, the heterogeneity of the masonry and its orthotropic behavior were neglected. The properties of an equivalent homogeneous isotropic material have been defined according to the strength and the Young's modulus values reported in the Commentary of the Italian Building code (CMTI 2019). To simulate the post-elastic behavior of the material, the concrete damage plasticity (CDP) model was chosen (Lubliner et al. 1989, Lee and Fenves 1998). At the beginning, this material model was conceived for concrete, later its use was also extended to masonry structures (Acito et al. 2016, Casolo et al. 2017, Valente et al. 2017, Bertolesi et al. 2018, Sarhosis et al. 2018, Valente and Milani 2018b). These materials have a similar brittle behavior, associated to two main failure mechanisms: cracking in tension and crushing in compression. Using the CDP, the mechanical properties of the material through the distinct uniaxial tensile and compressive constitutive laws can be defined. Moreover, the damage variables d_t (tensile) and d_c (compressive), functions of the plastic strain, reduce the undamaged elastic stiffness $\boldsymbol{D}_0^{el}$. The damaged elastic stiffness $\boldsymbol{D}^{el}$is defined as:

$$\boldsymbol{D}^{el} = (1-d)\boldsymbol{D}_0^{el}$$

$$\text{where: } 1-d = (1-s_t d_c)(1-s_c d_t)$$

Here, s_t and s_c are functions of the stress state and include the stiffness recovery effects associated with stress reversals. When the load changes from tension to compression, the compressive stiffness is recovered upon crack closure. On the contrary, once crushing micro-cracks have developed, the tensile stiffness is not recovered. Hence, s_t is equal to 1 and s_c is 0 when all the eigenvalues of the effective stress tensor are negative, otherwise equal to 1 when they are all positive. d_t and d_c are the two damage variables, respectively function of the tensile and compressive plastic strains. For the performed analyses the d_c parameter was assumed equal to 0, considering that the tensile strength of the material is significantly lower than the compressive strength in agreement with the examples in the Abaqus manual and other works (e.g. Valente and Milani 2018a). d_t is zero for elastic tensile strength and equal to 0.95 for the ultimate tensile strain. The model parameters were assigned according to the indications of the software documentation (Abaqus 2016) and to other similar applications and experimental studies (Van Der Pluijn 1993) (Table 3- 3).

Table 3- 3 Model parameters used for the performed analyses.

Φ	**ε**	**f_{b0}/f_{c0}**	**K_C**	**μ**
10°	0.1	1.16	2/3	0.002

Note: dilatation angle (*Φ*), the correction parameter of the eccentricity (ε), the strength ratio (f_{b0}/f_{c0}), the parameter (K_C) for the shape of the yield surface in the deviatoric plane, and the viscosity parameter (μ).

The post elastic behaviour in compression was defined through the definition of the compressive stress for different values of the inelastic strain $\tilde{\varepsilon}_c^{in}$ in agreement with the values reported by other authors (Table 3- 4). The inelastic strain in the uniaxial case is:

$$\tilde{\varepsilon}_c^{in} = \varepsilon_c - \frac{\sigma_c}{E_0}$$

In order to avoid mesh-sensitive results (De Borst 1997, Abaqus 2016), the post-elastic behaviour in tension was defined in terms of cracking displacements u_t^{ck} (Table 3- 4). The ultimate cracking displacement, at which strength is almost zero, is obtained considering a reasonable fracture energy G_f equal to 48 N/m (Bejarano-Urrego et al. 2018), which allows to study the complete evolution of damage, without convergence issues.

For the analyses, it was assumed that the arches and walls were made of different types of masonry. The properties of the masonry considered for the walls and arches are those reported in Table 3- 4, respectively referred to ''uncut stone masonry, with external leaves of limited thickness and infill core (three-leaf stone masonry)'' (muratura a conci sbozzati con paramento di limitato spessore e nucleo interno) and "cut stone masonry with good bond" (muratura in pietra a spacco con buona tessitura) (CMIT 2019). For the first type of masonry, a correction coefficient equal to 0.8 was considered, in agreement with the reported values in table C8A.2.2 (CMIT 2009) for masonry walls with a thick inner core and/or with poor mechanical properties.

Table 3- 4 Material properties for the masonry of the walls ("uncut stone masonry, with external leaves of limited thickness and infill core") and of the arches ("cut stone masonry with good bond") of San Salvatore church.

Walls (E =1152 MPa)		**Arches** (E =1740 MPa)	
Tensile behaviour		**Tensile behaviour**	
u_t^{ck} (mm)	σ_t (MPa)	u_t^{ck} (mm)	σ_t (MPa)
0.0	0.16	0.0	0.2
0.6	0.0016	0.5	0.002
Compressive behaviour		**Compressive behaviour**	
$\tilde{\varepsilon}_c^{in}$	σ_c (MPa)	$\tilde{\varepsilon}_c^{in}$	σ_c (MPa)
0.000	1.55	0.000	3.10
0.005	1.60	0.005	3.20
0.015	0.00	0.015	0.00

The same CDP model was used for the concrete of the roof of the 1997 configuration. In the absence of information on the type of concrete used, the properties of such material refer to NTC 2018 for a C25/30. The implemented values correspond to an elastic modulus of 31000 MPa, a compression strength (σ_{cu}) of 14.2 MPa, and a tensile strength equal to 1.2 MPa.

On the contrary, for the 2016 configuration with a timber roof, an elastic modulus parallel to the fiber equal to 10000 MPa, corresponding to softwood S2 (UNI 11035), was considered.

3.3.3.1 Frequency analyses

To understand the global dynamic behaviour of San Salvatore church, frequency analyses were performed. The same procedure was applied to both configurations, the 1997 and the 2016 ones, with concrete and timber roof respectively. Figure 3- 27 and Table 3- 5 show the most significant modal shapes of the 1997 configuration, the relevant natural periods and participating mass ratios for the longitudinal and transversal directions.

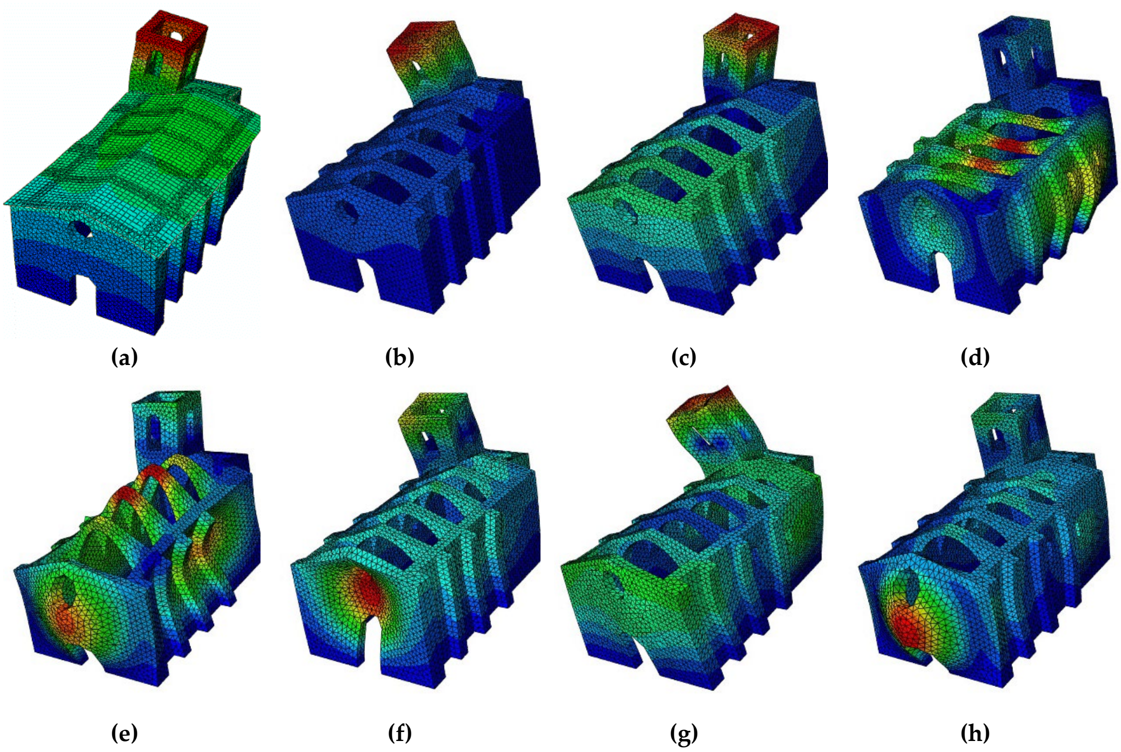


Figure 3- 27 Modal shapes of the 1997 roof configuration (concrete): **a.** Mode 1; **b.** Mode 2; **c.** Mode 3; **d.** Mode 4; **e.** Mode 5; **f.** Mode 6; **g.** Mode 7; **h**. Mode 8.

Table 3- 5 Synthesis of the most significant modes for the 1997 model with the relative periods of vibration and the participating mass ratio.

Figure of the modal shapes	Mode	Period (s)	Participating mass ratio, longitudinal direction (%)	Participating mass ratio, transversal direction (%)
3-28a	1	0.208	0.00	59.26
3-28b	2	0.161	24.82	0.09
3-28c	3	0.144	4.38	8.69
3-28d	4	0.113	0.07	0.12
3-28e	5	0.109	2.11	0.10
3-28f	6	0.108	34.51	0.42
3-28g	7	0.101	0.06	1.11
3-28h	8	0.091	8.17	0.30

The first mode involves the belfry and the nave in the transversal direction with a participating mass ratio of almost 60% (Figure 3- 27a). This is a high value for a masonry church, and it is due to the presence of the stiff concrete roof. The second mode has a displacement in the longitudinal direction, mainly concentrated on the bell tower with a participating mass ratio equal to almost 25 % (Figure 3- 27b). The third and the seventh modes show a displacement in the diagonal direction of the upper part of the bell tower and a rotation which principally involves the nave (Figure 3- 27c,Figure 3- 27g). The fifth mode is a vertical mode which causes the deformation of the arches due to the effect of compression and dilatation of the nave (Figure 3- 27e). The sixth and eighth modes are longitudinal modes which principally involve the façade and the bell tower with a light torsion (Figure 3- 27f; Figure 3- 27h).

Figure 3- 28 and

Table 3- 6 show the most significant modal shapes for the 2016 configuration (timber roof) with the corresponding period of vibration and participating mass ratio for longitudinal and transversal directions. In this configuration, the first significant mode is the 9th, while the first modes only activate locally the roof. Mode 9 involves the nave and it presents the highest participating mass ratio (40.49 %) in the transversal direction (Figure 3- 27a). Mode 11 follows the transversal direction and involves the bell tower and the connected triumphal arch of the apse (Figure 3- 27b). Mode 12 shows the highest participating mass ratio in the longitudinal direction with a value of almost 24%, involving the bell tower and the nave (Figure 3- 28c). Mode 13 (Figure 3- 27d) and Mode 25 (Figure 3- 27h) are torsional modes regarding the nave. Mode 14 reaches almost 20% of participating mass ratio in the longitudinal direction involving both the nave and the

façade (Figure 3- 27e). Mode 17 is interesting because it points out a compression effect of the nave in the transversal direction generating a vertical movement of the arches (Figure 3- 27f). Mode 22 principally shows a torsional mode of the bell tower (Figure 3- 27g).

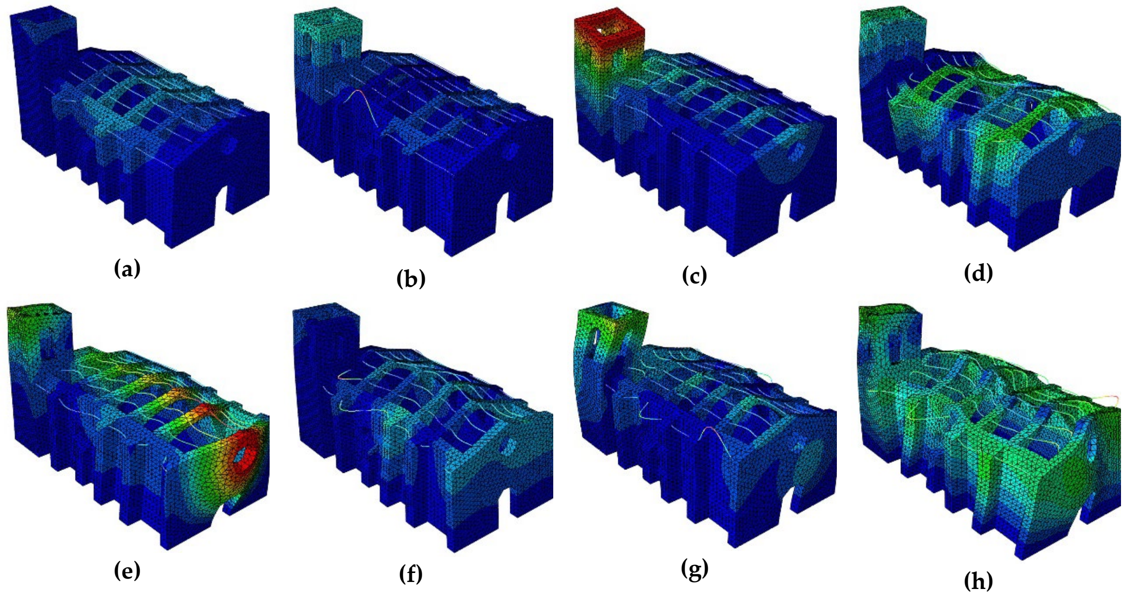


Figure 3- 28 Modal shapes of the 2016 roof configuration (timber roof): **a.** Mode 9; **b.** Mode 11; **c.** Mode 12; **d.** Mode 13; **e.** Mode 14; **f.** Mode 17; **g.** Mode 22; **h.** Mode 25.

Table 3- 6 Synthesis of the most significant modes of the 2016 model with the relative periods of vibration and the participating mass ratio.

Figure of the modal shapes	Mode	Period (s)	Participating mass ratio, longitudinal direction (%)	Participating mass ratio, transversal direction (%)
3-29a	9	0.257	0.02	40.49
3-29b	11	0.214	0.65	4.08
3-29c	12	0.187	23.70	1.64
3-29d	13	0.158	0.11	2.58
3-29e	14	0.130	19.50	0.03
3-29f	17	0.116	2.13	6.85
3-29g	22	0.098	0.69	2.23
3-29h	25	0.090	0.00	0.42

Table 8 synthesizes the influence of the roof type on the vibration modes of the church structure. For the concrete roof configuration, Mode 1 (Figure 3- 28a) is the principal transversal mode with a participating mass ratio of 60% and a period of 0.208 s; while, for the timber beam configuration, the highest participating mass ratio in the same direction appears in Mode 9 (Figure 3- 27a), with a period of 0.257s. The concrete roof makes the structure stiffer in the transversal direction, reducing the vibrating period of 20%. The same considerations can be formulated for the longitudinal direction: the main vibrating modes are Mode 2 (Figure 3- 28b) and Mode 6 (Figure 3- 28f) for the concrete roof configuration and Mode 12 (Figure 3- 27c) and Mode 14 (Figure 3- 27e) for the roof timber one. The corresponding periods for each mode, in the two configurations, show the stiffening due to the presence of the concrete roof, even if the percentage reduction is lower (14% and 17%) than in the transversal direction (20%).

Table 3- 7 Comparison between the principal vibration modes for the two configurations (concrete roof and timber roof).

	Concrete roof			Timber roof			Difference (*)	
Direction	**Mode**	**Period (s)**	**Participating mass ratio (%)**	**Mode**	**Period (s)**	**Participating mass ratio (%)**	**Period**	**Participating mass ratio**
Transversal	1	0.208	59.26	9	0.257	40.49	- 20 %	18.77%
Longitudinal	2	0.161	24.82	12	0.187	23.70	- 14 %	1.12 %
	6	0.108	34.51	14	0.130	19.50	- 17%	15.01%

(*) The differences are calculated considering the timber roof configuration as reference

3.3.3.2 Seismic analyses

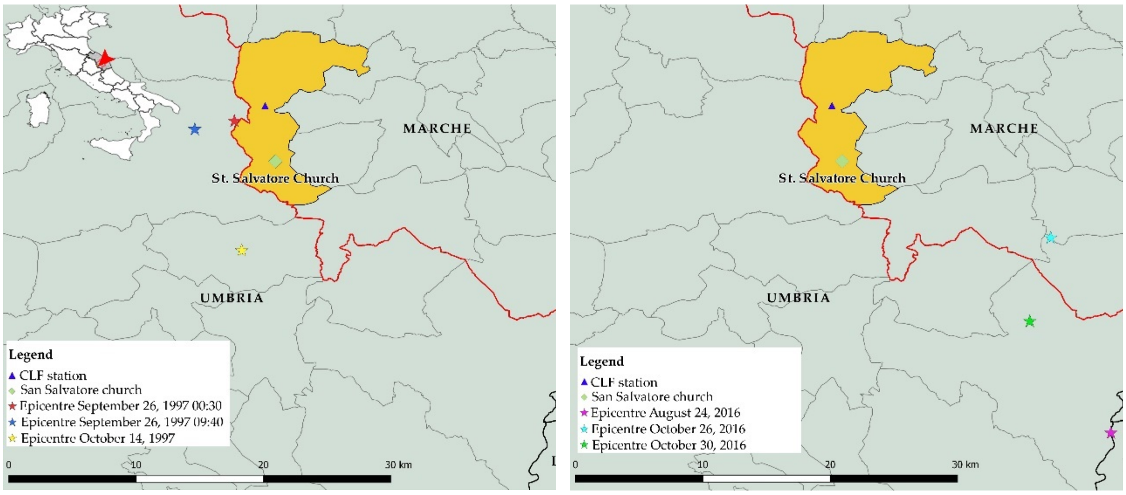
To study the seismic response of San Salvatore with the two roof configurations, the recorded data from the Colfiorito station (CLF) were used (Luzi et al. 2019). Table 3- 9 reports the data of CLF station.

Table 3- 8 Information on Colfiorito recording station (CLF) (Sferrazza Papa et al. 2019).

Station Code	Latitude	Longitude	Altitude (m)	Site Class	Morphology	Topography
CFL	43.03671	12.92043	701	D*	Valley centre	T1**

* It is referred to the soil classification expressed in Eurocode 8 (CEN 2005).
** It is referred to topographical categories presented in table 3.2.III of NTC 2018 (NTC 2018).

Figure 3- 29 shows the position of CLF station, the church and the epicenters of the 1997 Umbria-Marche and the 2016 Central Italy earthquakes, in order to contextualize the two episodes.



(a) (b)

Figure 3- 29 Localization map of the church (42.983453, 12.930303) in Serravalle di Chienti municipality (Yellow), the recording station (43.03671, 12.92043) and the epicentres of the major shocks of the two considered earthquakes: **a.** the 1997 Umbria-Marche earthquake; **b.** the 2016 Central Italy earthquake (Sferrazza Papa et al. 2019).

Table 3- 9 provides additional information on the main seismic shocks and the distances between the epicentres, the church and CLF station. The comparison of the data motivates to choose, for each one of the two seismic events, the event with the worst conditions. The two episodes, framed in red in the table, were chosen and they correspond to the shock of September 26, 1997 of the 09:40 a.m. and October 30, 2016.

Table 3- 9 Data on the principal shocks of the two studied earthquakes. This table includes the distances of the recording station and the church from the epicentres, the depth and the magnitude for each seismic event (Sferrazza Papa et al. 2019).

Seismic events	**The 1997 Umbria-Marche earthquake**			**The 2016 Central Italy earthquake**		
	14 Oct. 15:23	**26 Sept. 00:33**	**26 Sept. 09:40**	**24 Aug.**	**26 Oct.**	**30 Oct.**
Distance epicentre from the church (km)	10.19	6.10	8.73	40.95	26.24	23.72
Distance from the station (km)	15.60	3.84	7.96	45.49	27.16	31.04
Depth (m)	6.00	5.70	5.70	8.1	7.5	9.20
Magnitude (Mw)	5.65	5.70	6.01	6.0	5.90	6.5
Distance CLF station from the church (km)	6.00					

Figure 3- 30 shows the accelerograms registered in the Colfiorito station for the three components (NS, EW, and Z).

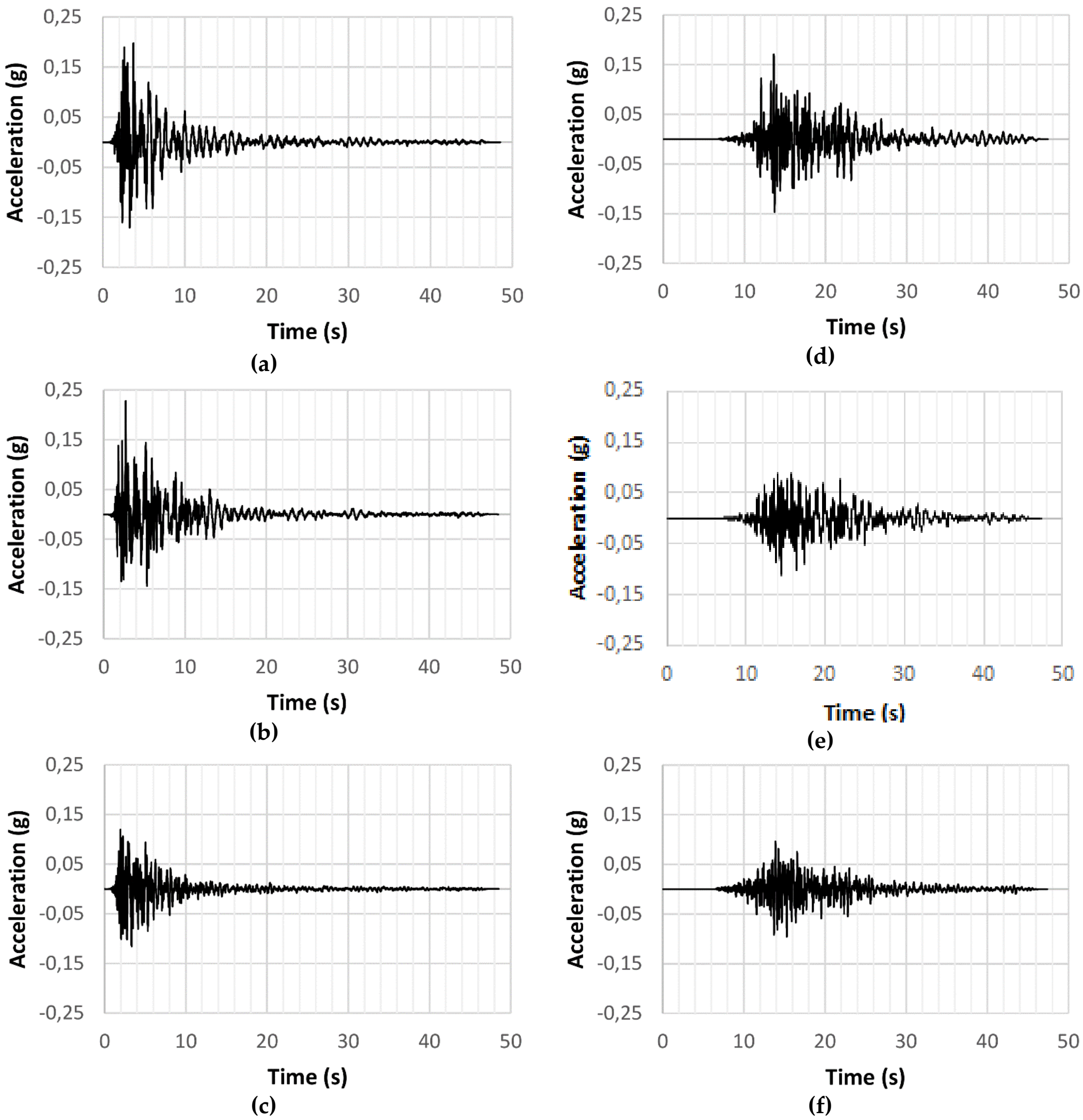


Figure 3- 30 Processed accelerograms registered in the Colfiorito station during the two considered events. The data refer to the earthquake which occurred on September 26, 1997 (09:40 a.m.) for the three components: a. North-South (NS); b. East-West (EW); c. Z. The data report the earthquake which occurred on October 30, 2016 for the three components: d. North-South (NS); e. East-West (EW); f. Z (Sferrazza Papa et al. 2019).

The accelerations, recorded in Colfiorito station, were directly applied at the base of the church because no recording nor soil data were available for the site of the church. Using a GIS software, Figure 3- 31 was elaborated overlaying the PGA isocurves of the 2016 event, the position of the CLF station, the church, and the epicentres. The CLF station and the church are between the 12 % g and 18% g PGA isocurves. Consequently, applying at the base of the church the acceleration recorded in Colfiorito station was considered acceptable.

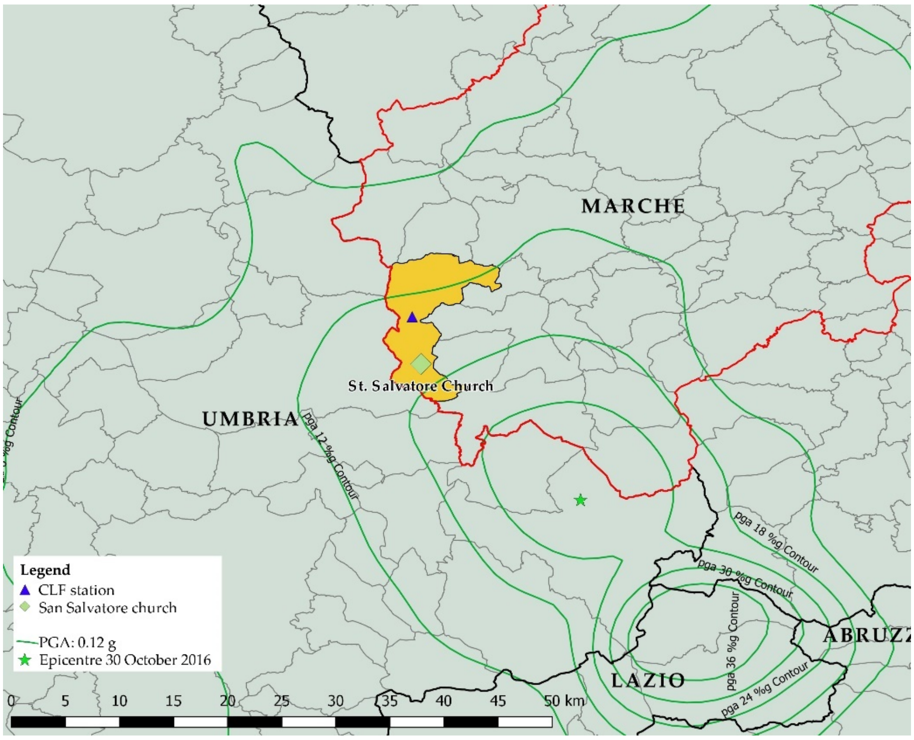


Figure 3- 31 This map shows the PGA isocurves of October 30, 2016 (12% g) with the position of the church and CLF station (Sferrazza Papa et al. 2019).

In order to characterize the seismic events considered in the analyses, the elastic response spectra, in terms of acceleration, have been compared with those obtained with the standard procedure of the National Building Code (NTC 2018). This procedure considered both the Life-Safety Limit State (SLV) and the Damage Limit State (SLD) spectra, with return periods of 475 and 50 years respectively, referring to soil category D, that is the same of Colfiorito station (Figure 3- 32).

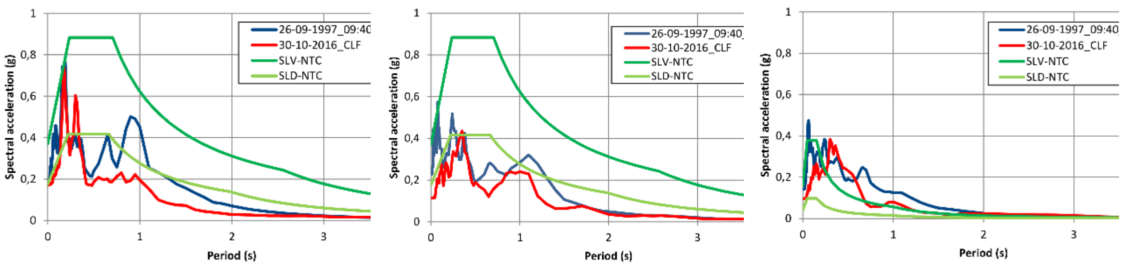


Figure 3- 32 Comparison of the elastic response spectra, in terms of acceleration, of the two events with the same spectra from the National Building Code: **a.** North South direction (NS); **b.** East-West direction (EW); **c.** Z direction (Sferrazza Papa et al. 2019).

Table 3- 10 reports the peak ground acceleration, PGA, and the Arias intensity, I_a, (Arias 1970) for the three directions. In the table, T90 is the effective duration of the ground

motion excerpted from the ITACA database (Luzi et al. 2019), defined as the time required to pass from the 5% and the 95% of the Arias intensity.

Table 3- 10 Seismic data of the September 26, 1997 and the October 30, 2016 earthquakes (Sferrazza Papa et al. 2019).

Seismic event	September 26, 1997 09:40	October 30, 2016
PGA_{NS} (m/s²)	1.938	1.676
PGA_{EW} (m/s²)	2.234	1.107
PGA_{Z} (m/s²)	1.172	0.949
I_{a_NS} (m/s)	0.457	0.369
I_{a_EW} (m/s)	0.363	0.254
I_{a_Z} (m/s)	0.162	0.155
T90 (s)	11.445	18.695

Figure 3- 32 and Table 3- 10 show the peculiarities of the two seismic events, pointing out some interesting aspects. The 1997 earthquake has comparable peak accelerations in the two directions (NS, EW) and high values in the vertical one (Z). On the contrary, the 2016 seismic event is characterized by higher values of accelerations in the North-South direction, but always lower than those of 1997. The duration of the seismic shock, defined as T90, is also a representative value: in the case of the 2016 seismic event, it had a higher value than in 1997. Figure 3- 32 shows that both events are under the SLV code spectrum, with the only exception of the Z components. In particular, for the period values lower than 0.1 s and higher than 1 s, the 1997 event shows higher spectral acceleration than the 2016 one, otherwise they are comparable.

Non-linear implicit dynamic analyses were performed with both configurations (concrete roof, timber roof), using the accelerograms specified in the section. Each analysis took about 4 hours, they were executed on a computer with a Linux operating system, a 4 cores Intel Core i7-6700K CPU and 32 GB RAM. The maximum size of the time step was set at 0.005 s, but, at some instants, the analyses required the reduction up to 2.5×10^{-5} s for convergence.

The results of the analyses are reported in terms of damage maps of the computed tensile damage d_t. The September (1997) shock was applied to the 1997 configuration and the damage of the triumphal arch (Figure 3- 33a), the façade (Figure 3- 33b), the bell tower (Figure 3- 33c), and the apse (Figure 3- 33d) was obtained. No damage in the buttresses and lateral walls emerged from the analyses. The damage in the buttresses (Figure 3-

20d) is probably due the performed rebuilding at a different time, while the lateral walls (Figure 3- 20c) suffered disintegration of the external leaf. These types of damage were not obtained in the analyses because they are due to local characteristics difficult to include in a global model.

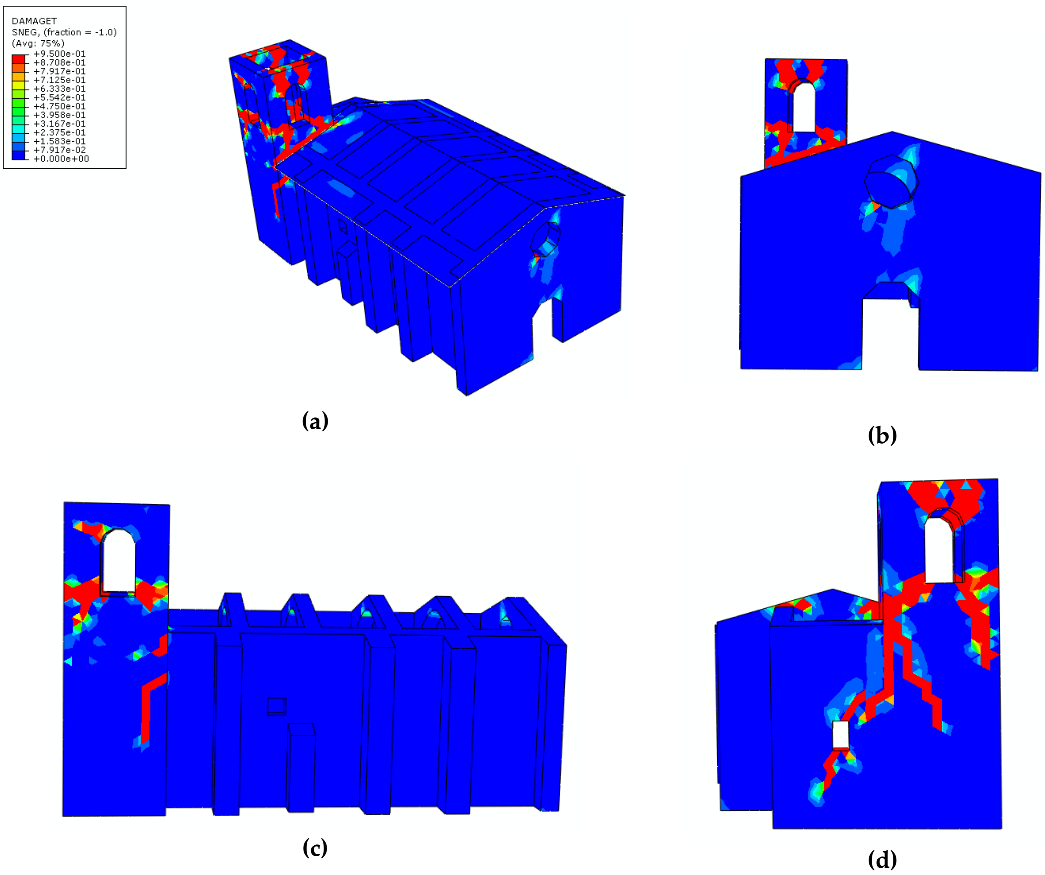
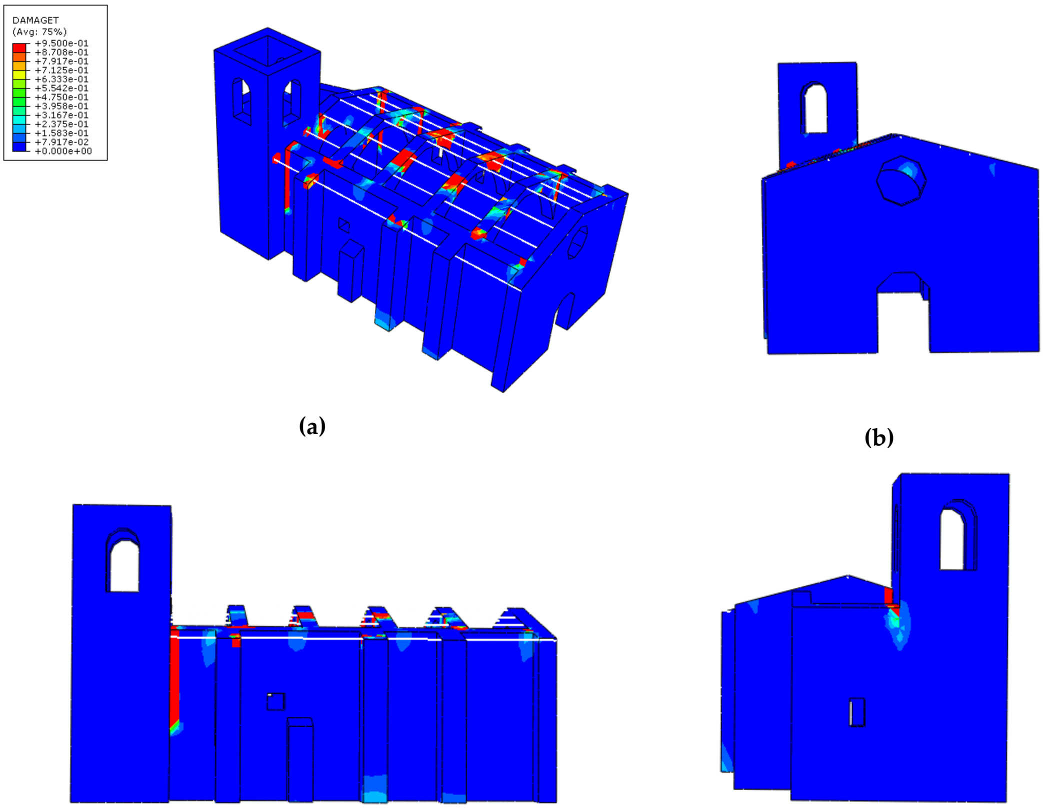


Figure 3- 33 Results of the seismic analyses applying the earthquake which occurred on September 27, 1997: **a.** North west view; **b.** West view; **c.** North view; **d.** East view.

The accelerogram selected for the 2016 Central Italy event was applied to the 2016 configuration. Figure Figure 3- 34a shows the damage at the base of the longitudinal nave in the interior side of the south wall and in the arches, where the timber beams of the roof enter the masonry. It also shows a local damage in the façade at the intersection with the transversal walls of the nave, without showing any damage in the rest of the façade (Figure 3- 34b). The damage between the bell tower and the rest of the structure of the church in the east and north side (vertical crack) was also observed (Figure 3- 34c;Figure 3- 34d).

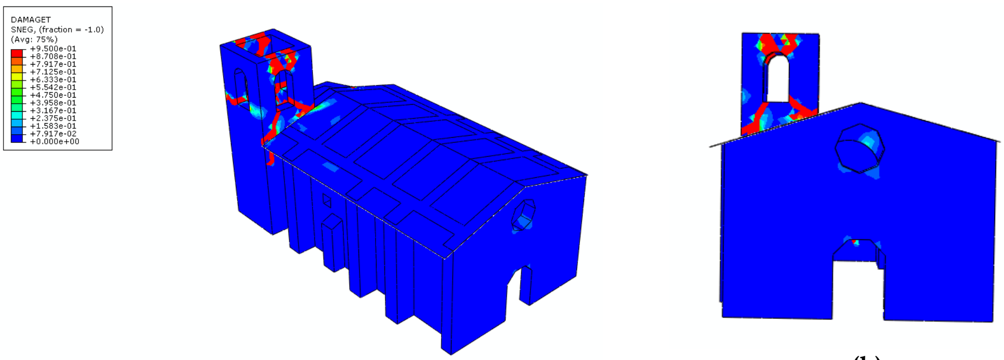


(a) (b)

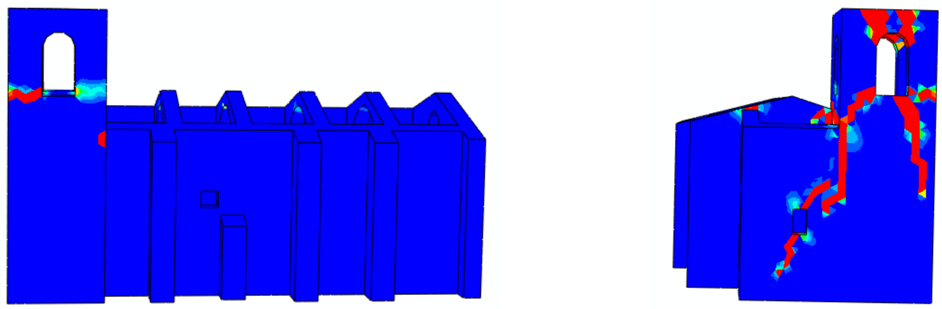
(c) (d)

Figure 3- 34 Results of the dynamic analyses for the 2016 configuration applying the occurred earthquake (October 30, 2016): **a.** North west view; **b.** West view; **c.** North view; **d.** East view.

The models were able to represent the damage correctly. So, to further investigate the effect of the two intervention modalities, the analyses were repeated, applying to the 1997 configuration the 2016 earthquake (Figure 3- 35) and to the 2016 configuration the 1997 earthquake (Figure 3- 36).

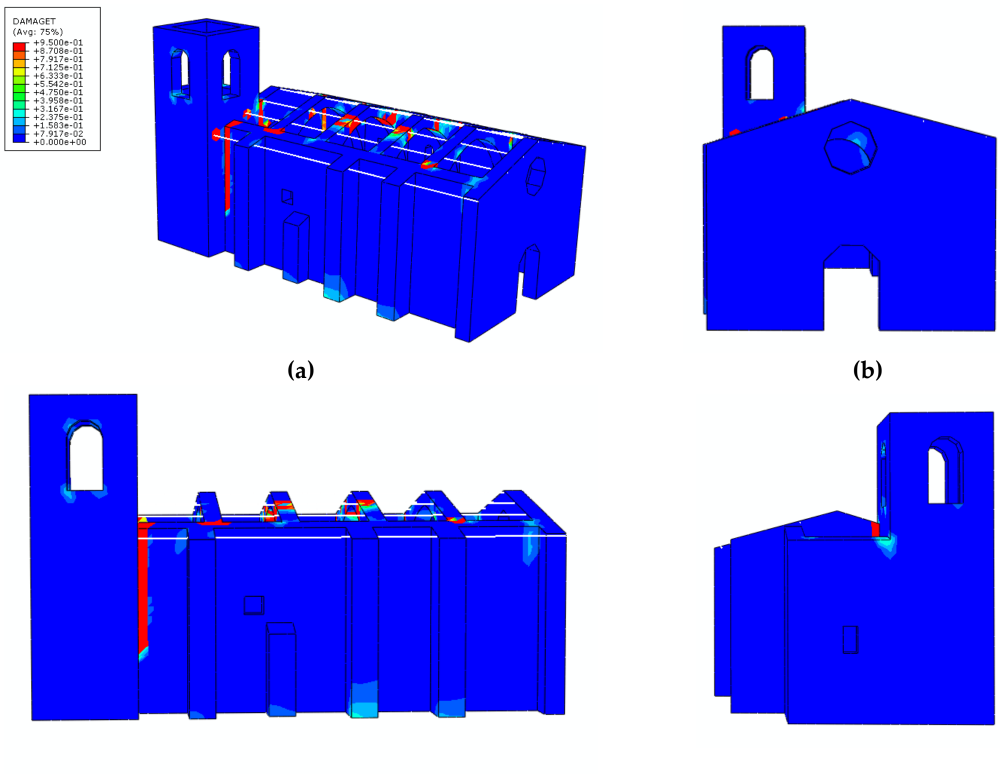


(a) (b)



(c) (d)

Figure 3- 35 Results of the seismic analyses for the 1997 configuration applying the October 30, 2016 earthquake: **a.** North west view; **b.** West view; **c.** North view; d. East view.



(c) (d)

Figure 3- 36 Results of the seismic analyses for the 2016 configuration applying the September 27,1997 earthquake: **a.** North west view; **b.** West view; c. North view; **d.** East view.

From the comparison of Figure 3- 35 and Figure 3- 36 with Figure 3- 33 and Figure 3- 34, the damage pattern appears independent from the characteristics of the seismic events. At this point, some considerations on the influence of the roof type can be expressed: on the one hand, the concrete roof guarantees the box-like behavior, but it caused a damage principally concentrated on the façade and on the bell tower (Figure 3- 33a;Figure 3- 33b); on the other hand, the timber roof produced local damage, concentrated on the arches and on the intersection between the transversal walls and the façade (Figure 3- 33;Figure

3- 35;Figure 3- 34;Figure 3- 36). Moreover, an independent structural response of the bell tower was observed in the 2016 configuration. In this case, the timber roof is not capable to tie together the structure of the church and of the bell tower, causing less damage in the apse and bell tower, but producing a pounding effect in the part of connection between the apse and the bell tower, and between the lateral wall of the nave and the bell tower.

Finally, the comparison of the displacement-time history of some points in the two church configurations for the September 27, 1997 earthquake was performed (Figure 3- 37).This comparison confirms how the change of the roof configuration significantly modifies the response of the bell tower, with a significant reduction of the displacements and of the residual displacements with the timber roof (Figure 3- 37a; Figure 3- 37b). For the façade, the concrete roof, on the one hand, reduces the displacements in the gable (Figure 3- 37c) and, on the other, increases the out-of-plane displacements at the centre of the façade (Figure 3- 37d). About the nave, the timber roof limits the constraint on the lateral walls and for this reason they tend to bend horizontally more than in the 1997 configuration; this also justifies the occurred damage in the corners of the lateral walls. The residual displacement is limited, due to the tie rods and the buttresses that constrain this movement (Figure 3- 37e).

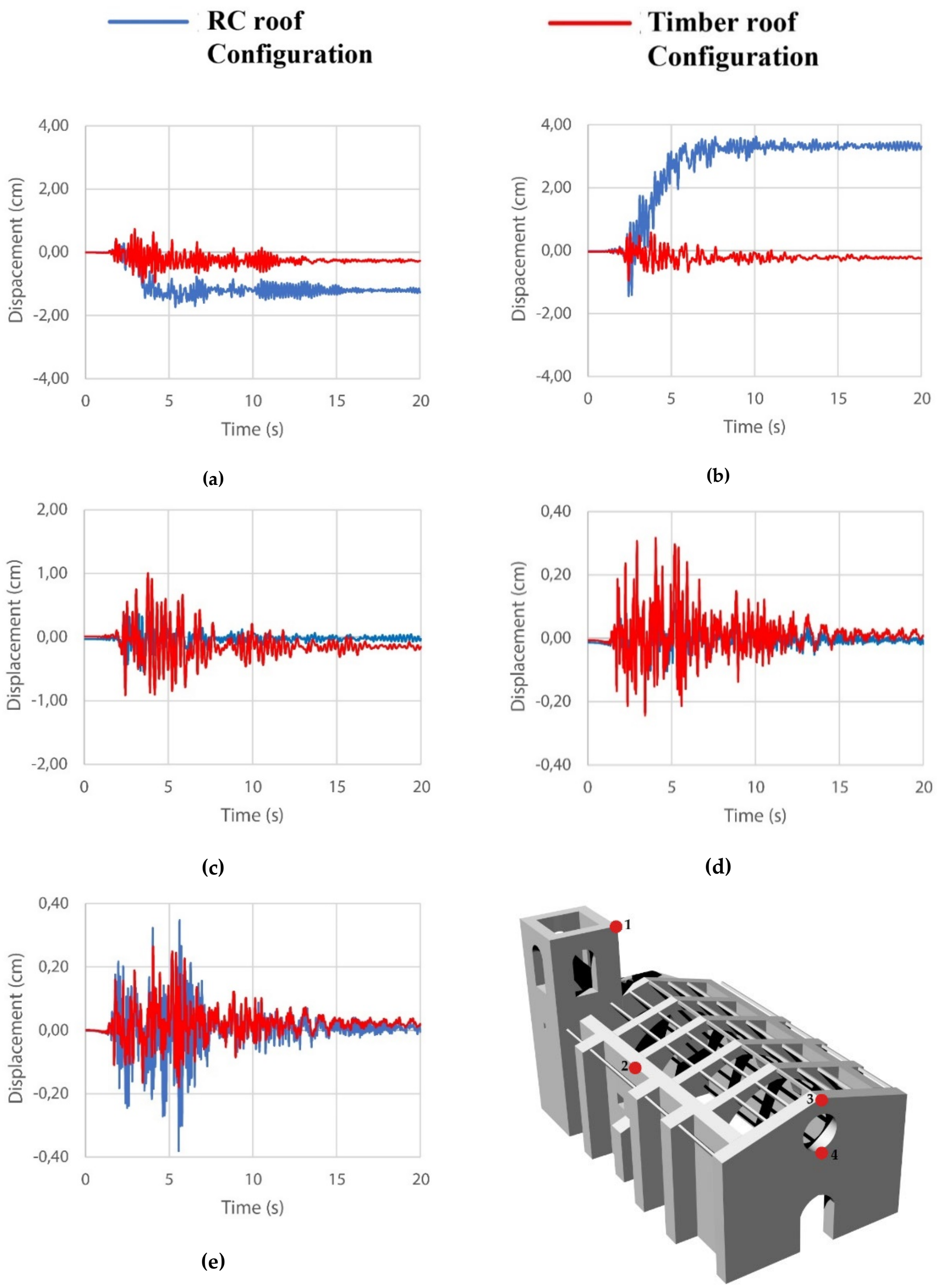


Figure 3- 37 Comparison of the displacement-time history of the two church configurations (blue the 1997, red the 2016) subjected to the September 27,1997 earthquake. For the comparison it was plotted the displacement for the following points: **a.** point 1 taken in the bell tower for the longitudinal direction; **b.** the transversal direction; **c**. point 2 in the lateral wall; **d.** point 3 in the gable for the out-of-plane displacement; **e.** point 4 in the centre of the façade out of plane (Sferrazza Papa et al. 2019).

3.4 TERRITORIAL ASPECTS FROM THE SAMPLE

After analyzing the damage produced by the Central Italy earthquake of 2016 on a sample of churches and developing in depth a specific case study, it seems possible to sort out for these buildings some *territorial specificities*, summarized in the following.

In the churches examined, two construction materials are principally used in the area, the brick and the stone masonry. When the wall is made of bricks, the cracks follow the path of the mortar. Stone walls are made of three leaves and they appear subjected to leaf separations. This was the case observed for example in San Salvatore after the 1997 Umbria-Marche earthquake.

The vault, built as a thin flat-brick shell, confirmed to be a particularly vulnerable part of the building. At the same time, this particular element is usually continuous and being in contact with the façade, suffered damage also due to the vibration and pounding effect of the façade wall when this last became not sufficiently restrained by the longitudinal walls of the nave or longitudinal tie rods. The absence of stiffening ribs makes the thin brick vaults of this area more subjected to damage compared to the same type of vault in Lombardy area. The same is also due to the characteristic that they tend to be continuous with arches that separate the portion of vaults.

The interventions performed after the 1997 Umbria Marche earthquake showed in general their effectiveness in modifying the vulnerability of the element. In some cases, they limited the damage and influenced its path, as a consequence of the increase in stiffness they provided. Two examples are the steel hoop in S. Maria Nuova, that protected the dome relocating the damaged area elsewhere, and the concrete roof before 1997 in the church of San Salvatore that provided a transversal link, with the side effects that have been discussed in 3.3.3.2.

The tie-rods have demonstrated to be an effective seismic provision, following the philosophy of the minimum intervention. In the case of San Francesco their presence was determinant in helping the response of the nave and avoiding the overturning of the longitudinal walls of the nave.

In similar churches of this geographical area, masonry reinforcement ribs were found at the extrados of the thin brick vaults to make them stiffer.

Façades suffered more damage when presenting a soaring gable. Their configurations with rose windows and openings make shear damage more common in this macro-element, and if close to the end of the plane of the façade they make the 'v' shape overturning of the upper portion easy to develop.

Depending on the history of construction, the bell tower sometimes is annexed to the triumphal arch. Such condition justified the increase of the vulnerability for the triumphal arch, that suffered damage when the bell tower was solicited by lateral forces. This aspect has been pointed out in particular when the box-like behaviour of the structure is more guaranteed, as in the case of San Salvatore. Belfries in this area were found either constructed with a different fabric or, other times, with different materials. The resulting discontinuity is reflected in the behaviour, offering a different response to seismic actions.

CHAPTER 4

EASTERN LOMBARDY: THE 2012 PIANURA PADANA EMILIANA EARTHQUAKE

This section presents cases from another geographical area of Italy, different in seismicity, materials and construction techniques. This area was chosen because it exemplifies a place with low seismic awareness. The recent earthquake, which struck the area in 2012, has strongly affected not only the industrial building stock, but also the churches. The damage scenario has provided useful information to study the church response in relation to the specific construction practices of this territory. The information on the damage occurred to churches comes for the seismic damage archive in the Superintendence of Cultural Heritage in Brescia. A group of churches with different levels of damage were selected. A case study was chosen from the sample in order to investigate in more detail the seismic vulnerability associated to the baroque typology that is largely present in the area concerned. Finally, some territorial aspects are pointed out in the perspective of building up knowledge on the seismic vulnerability of churches in relation with the construction culture of this territory.

This chapter is devoted to an Italian area, the province of Mantova in Eastern Lombardy, where a relatively low seismicity level lead to underestimation of the risk. The 2004 Salò and the 2012 Pianura Padana earthquakes events somehow startled most of the people and practitioners. Considering the low seismicity of the area, in the interpretation of the damage that affected the church building stock, it is helpful to elaborate some considerations on the territorial specificity. In particular, in this chapter the reference is the 2012 Pianura Padana earthquake. The area of Mantova close to the Emilia Romagna region, called 'basso mantovano', is rich of churches. According to a report issued after the 2012 reference earthquake by the dioceses of Mantova, 129 out of the 302 church buildings of the dioceses (i.e. 42 %) were damaged.

This chapter contributes to elaborate the proposed approach by observing the construction characteristics specific of this area that have shown an influence on the structural response in the examined sample of churches.

The baroque church typology common in this area, with its recurrent post-earthquake damage patterns and elements of vulnerability, was the object of this study. The distinguishing baroque characteristics are the plan configuration and the façade configuration, generally three-partitioned, and the system of vaults and arches. San Bartolomeo in Quistello was selected as a case study.

4.1 The seismicity of the area

The churches selected for the present study are in the Mantova OltrePo, which is the last stretch of Lombardy close to the border of Emilia and the Po river. The classification of the area as seismic became effective with the (OPCM 2003). Consequently, seismic protection provisions were not routinely used before and some retrofitting interventions, such as concrete slabs or rigid ring beams, largely used in other regions with a higher and recognised seismicity, as for example observed in Central Italy (Tobriner et al. 1997, Modena and Binda 2009) are not typical of this area. However, some construction details, as the presence of tie rods in masonry buildings, seem to indicate a historical awareness of the seismicity for the area. At this regard, Sorrentino et al. (2014), quoting (Guidoboni 1987), report that the damage occurred after the 1570 earthquake, in the close province of Ferrara, motivated the development of one of the earliest designs for an earthquake-resistant house, suggesting the use of the tie rods (Sorrentino et al. 2014). Figure 4- 1 shows the seismic hazard for the Lombardy region in terms of acceleration for a probability of exceedance of 10% in 50 years. The province of Mantova has been highlighted in the figure. The province of Mantova has values of peak ground acceleration in the range of 0.175-0.150 g close to the border of Brescia and 0.100-0.125 g close to the border of Emilia.

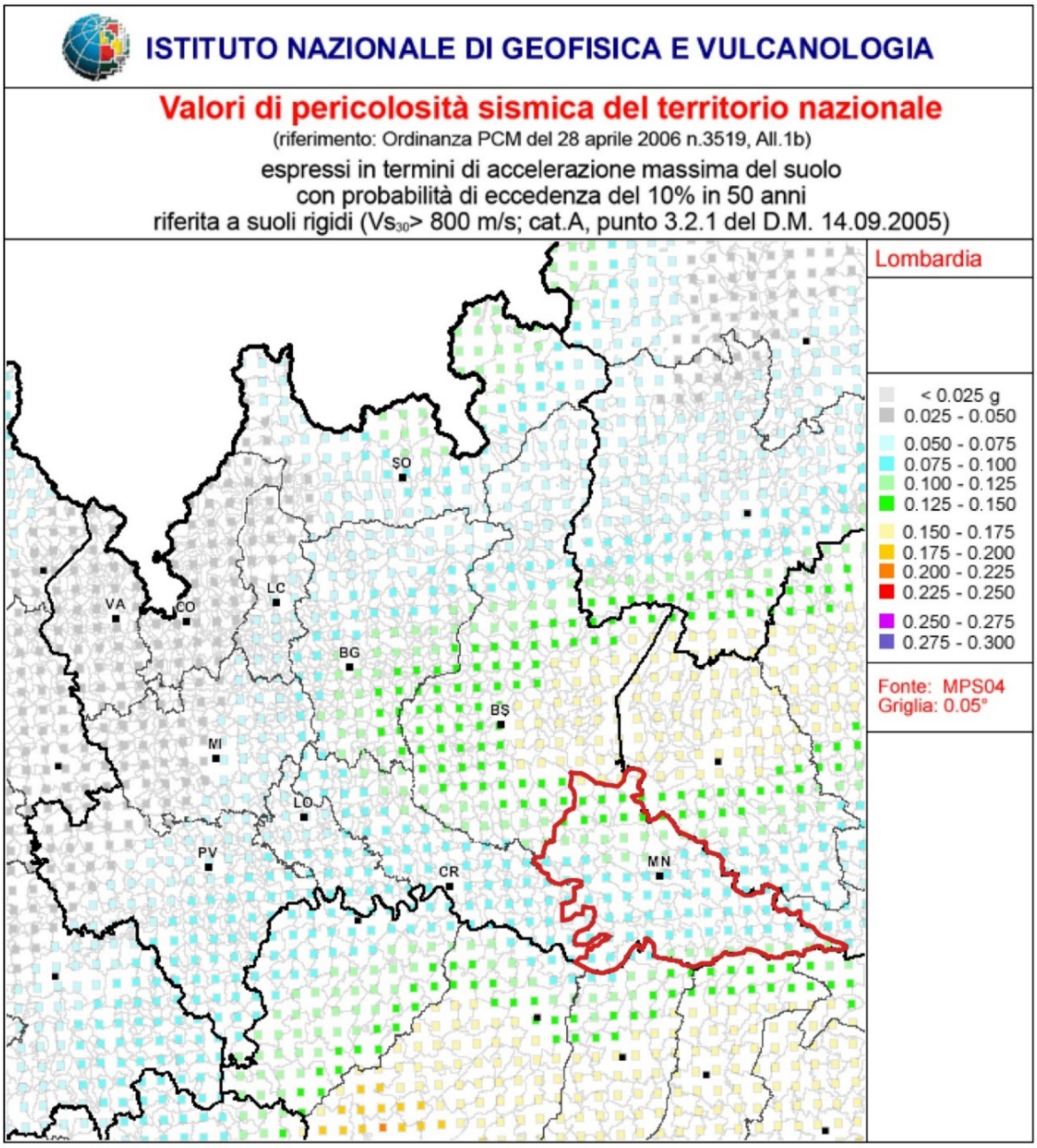


Figure 4- 1 Seismic hazard in Lombardy, for a probability of 10% in 50 years.

According to the Italian Macroseismic Database (Locati et al. 2019) the province is characterized by a moderate-low seismicity, that is, a seismicity considerably lower than in the Central and Southern Apennine, Calabria, Oriental Sicily, and North-Eastern Italy.

4.2 The 2012 Pianura Padana Emiliana Earthquake: the two major shocks

In 2012, two strong seismic events interested the area: one on May 20th (5.9 M_L) with epicenter in Finale Emilia and another on May 29th (5.8 M_L) with epicenter close to Mirandola in the province of Modena. They were generated by the three principal seismogenic units (Poggio Rusco-Migliarino, Finale Emilia-Mirabello, and Carpi-Poggio Renatico) which cross the area where the churches, treated in sections 4.2.1, are located (Figure 4- 2).

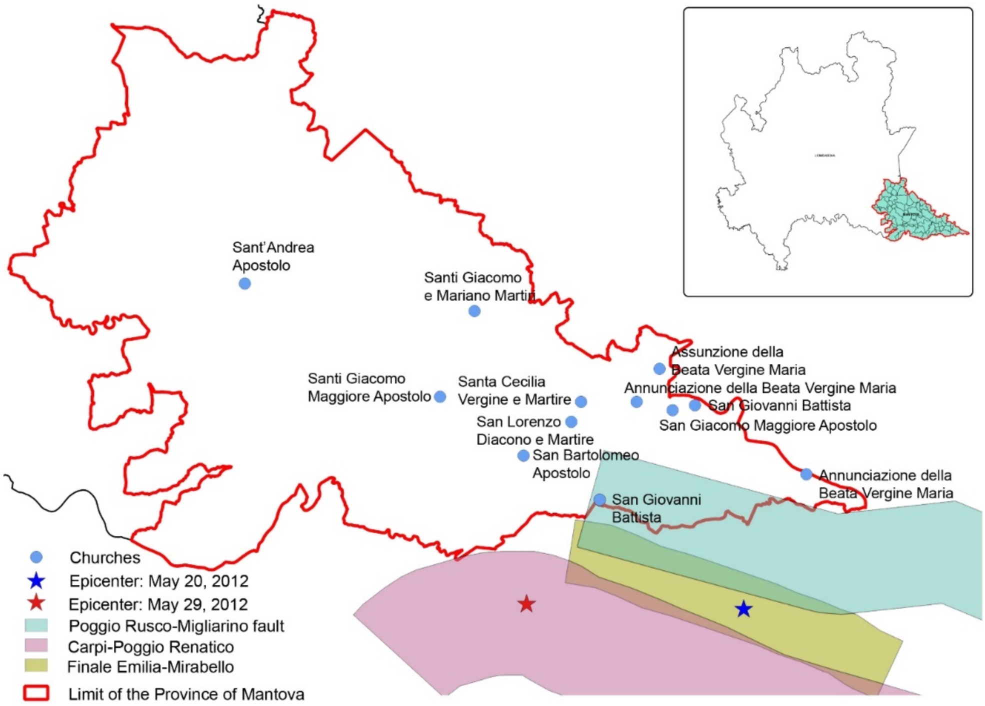


Figure 4- 2 Epicenters of the two major shocks (blue and red stars) with the churches of the sample (blue dots) and the three seismogenic units of the area (red, green, blue stripes) (DISS Working Group, 2018).

The first main shock was preceded by a milder shock on May 19 (4.1 M_L), then, in the days between the shocks of the 20th and 29th of May, thousands of aftershocks and minor earthquakes involved a large area that included the provinces of Modena, Ferrara, Rovigo, and Mantova (ISIDe). In such short period, several historical masonry buildings were damaged (Cattari et al. 2012; Parisi and Augenti 2013, Penna et al. 2014), as well as industrial sheds, causing the interruption of economic activities and determining a high economic impact for this geographical area. An impressive damage after the 29th May shock was the collapse of the clock tower and the fortified tower in Finale Emilia (Acito et al. 2014). The churches, as well, were strongly affected (Sorrentino et al. 2014, Valente et al. 2017, Valente and Milani 2018a, Indirli et al. 2012). According to Bignami et al. the seismic events were caused by compressional faulting over blind thrusts of the western Ferrara Arc and phenomena of liquefaction were also observed (Bignami et al. 2012).

4.2.1 The sample of damaged churches

The 2012 Pianura Padana Emiliana earthquake damaged several churches in the area under study, out of which 11 have been selected. They belong to some of the 15 municipalities of the province of Mantova that were part of the affected area, that is, Felonica, Gonzaga, Moglia, Magnacavallo, Motteggiana, Pegognaga, Poggio Rusco, Quingentole, Quistello, San Benedetto Po, San Giacomo delle Segnate, San Giovanni del Dosso, Schivenoglia, Sermide and Villa Poma (Figure 4- 3).

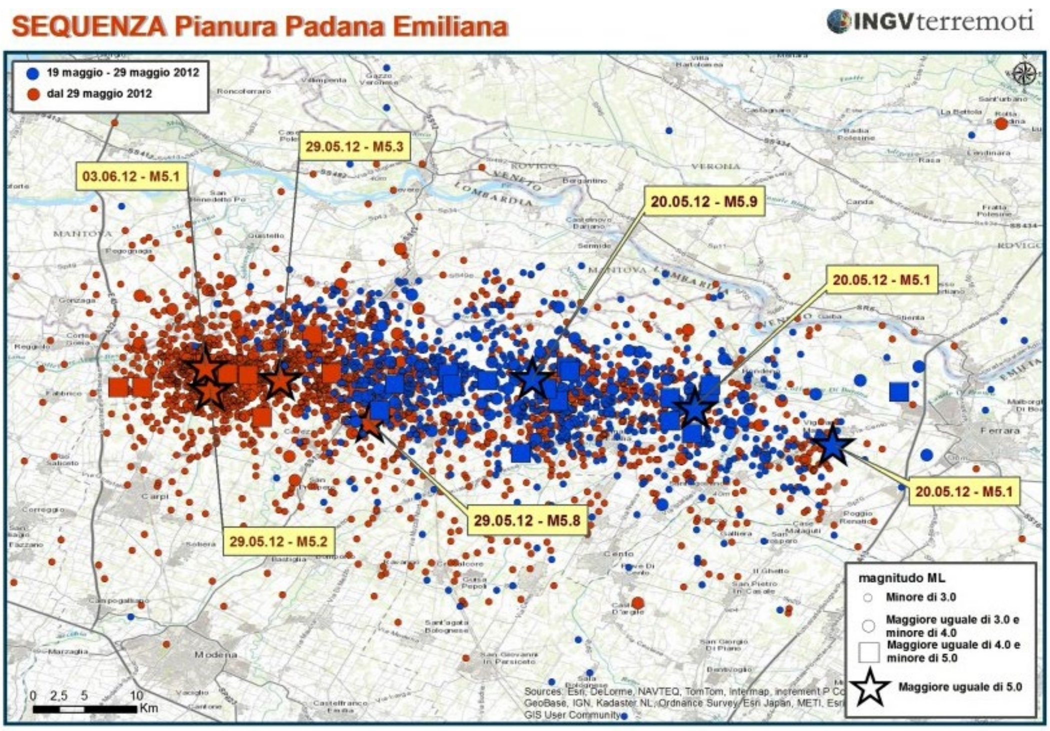
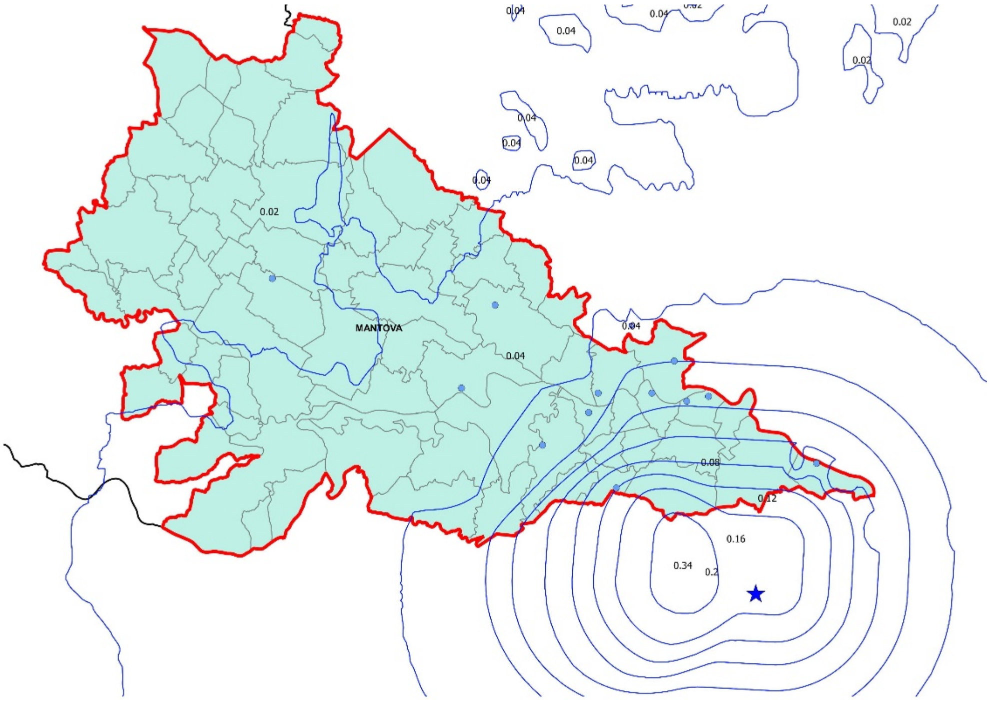
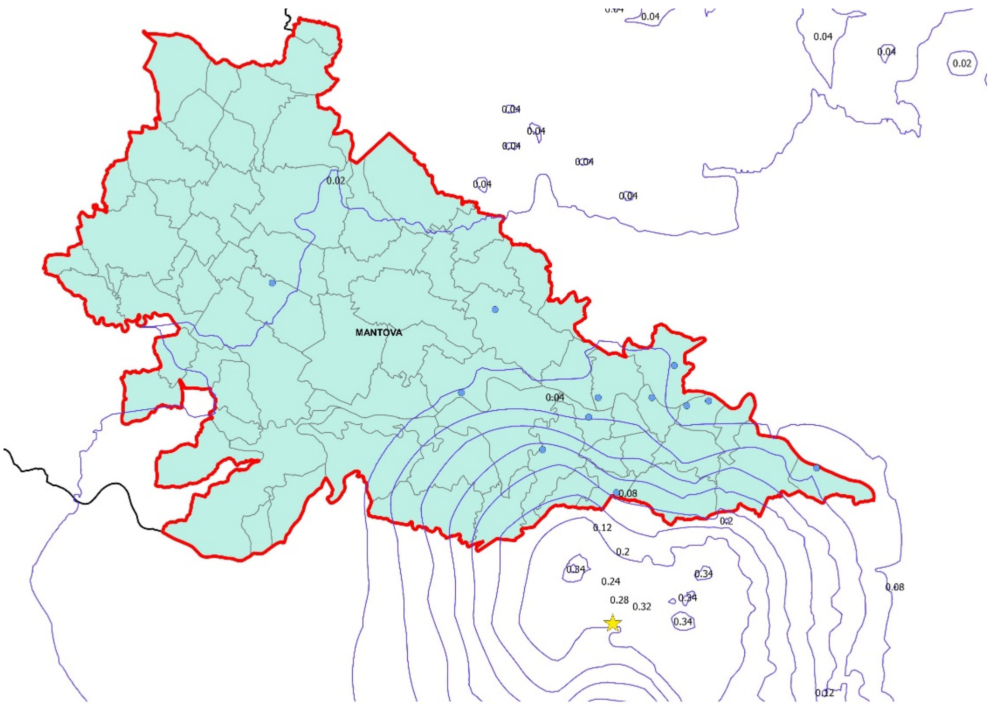


Figure 4- 3 Area affected by the 2012 Pianura Padana Emiliana earthquake (INGV 2017).

For the OltrePo area, Figure 4- 4a shows the peak ground acceleration of the first shock and the location of the churches of the sample, while Figure 4- 4b reports the PGA of the second event. The magnitude was significant in both cases: 5.9 M_L on May 20 and 5.8 M_L on May 29.



(a)



(b)

Figure 4- 4 Isocurves of PGA of the main events with indication of the church sample (blue dots): **a.** event of May 20; **b.** event of May 29.

A damage survey campaign was carried out for some of the churches after the two shocks; not all the churches were checked after the first due to the short lapse of time between the two strong events and the thousands of aftershocks between them. Evolution of the damage was reported for some of the churches that sustained damage without collapse after both events. An example is the case of San Giovanni Battista in San Giovanni del Dosso with the total collapse of the gable during the second shock, already severely damaged after the first one (Figure 4- 5).

(a)

(b)

Figure 4- 5 Damage to the gable of San Giovanni Battista (Superintendence of Brescia 2018): **a.** Damage of the gable after the first shock (May 20), with a level of damage equal to 3 according to EMS-98 scale for Mechanism 2 (Mechanism of the gable) in the abacus of mechanisms; **b.** Collapse of the gable after the second shock (May 29), damage level 5.

Another example is the case study considered here for this geographical area, San Bartolomeo church in Quistello, which is going to be analyzed more in detail in Section 4.3. The shock of May 20 caused the collapse of the marble element supporting the cross and the consequent damage of the vault adjacent to the façade. After the second seismic event, the compromised vault completely collapsed.

Figure 4- 6 shows the façades of the selected churches in order to understand the principal constitutive characteristics that could influence the seismic vulnerability of this macro-element.

(a) (b)

(c) (e)

(f) (g)

(h)

(i)

(l)

(m)

(n)

Figure 4- 6 The churches of the sample (Superintendence of Brescia 2018): **a.** San Lorenzo Diacono e Martire in Quingentole; **b.** Annunciazione della Beata Vergine Maria in Revere (Photo from wikimedia Commons); **c.** Sant'Andrea Apostolo in Sarginesco; **d.** San Bartolomeo Apostolo in Quistello; **e.** San Giovanni Battista in San Giovanni del Dosso; **f.** San Giovanni Battista in Borgofranco sul Po; g. Santa Cecilia Vergine e Martire in Lobiola; **h.** San Giacomo e Mariano Martire in Villa Garibaldi; **i.** San Giacomo Maggiore Apostolo in Bonizzo; **l.** Assunzione della Beata Vergine Maria in Ostiglia; **m.** San Giacomo Maggiore Apostolo; **n.** Assunzione della Beata Vergine Maria in Felonica.

In most of the cases, the façade is divided in three parts corresponding to the internal nave distribution. Pilasters give the rhythm to the façade configuration. Due to the use of the brick as material of construction, they are built adding a wythe of bricks. This constructive detail provides a thicker section for the façade which, from a structural perspective, in combination with the brick pilasters guarantee a homogeneity of material and technique, and thus of structural response. Three doors directly give access to each nave. At the same time, vertically, the façades are subdivided in two levels and a gable by two cornices. From the sample of churches, the highly soaring gable was observed to be a common characteristic of the façade especially for those of the baroque style (Figure 4- 7). Frequently, the churches of this geographical area have the gable standing above the end of the roof, even if the shape could change (Figure 4- 7). A central window, located in the second order above the cornice, gives light to the central nave. In some cases, the opening was closed by infilling it at a second time.

(a) (b)

(c)

Figure 4- 7 Example of soaring gable from the church sample (Superintendence of Brescia 2018).

Pinnacles are another non-structural element common in the investigated churches. Due to their position in the upper part of the façade and being not anchored, in case of earthquake they can constitute an element of vulnerability for the building.

Another peculiar characteristic of the churches of this territory is the use of brick as construction material. Brickwork is employed for the walls, the arches, and the vaults. The curtain walls are made of thin bricks with mortar joints set to constitute a regular pattern (Figure 4- 8).

Figure 4- 8 Example of brick pattern found in one of the church sample (Superintendence of Brescia 2018).

The brick pattern is often visible from the lateral walls of the church, where they were are not covered with plaster.

The bell tower is usually located on one side of the façade (i.e. Figure 4- 6h, Figure 4- 6n) or at the back of the church; in some cases they are annexed to the triumphal arch in proximity of the apse area (i.e. Figure 4- 6a, Figure 4- 6f). In some churches, the bell tower is part of the same structure of the church while in others it is independent (i.e. Figure 4- 6c).

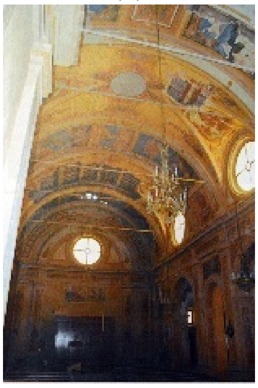
Another element frequently found and typical of these churches is the triumphal arch, which is generally confined by a couple of walls that contribute to its in-plane response. In a few cases, among the sample, there are some vaults or domes made of reeds. The most common practice for vaults, however, was the use of vertical stacked bond, forming thin shells. This technique found a large use due to the fast realization and the small quantity of material needed (about 1/3 less of material compared to the construction of the 'voissoir' vaults). Depending on the geographical areas, such technique has shown an adaptation, both for the dimensions of bricks and the construction technique (for more detail see Annex Ic). Such technique is found frequently in churches of this area. Figure 4- 9 shows the interior of churches with different vault geometries, even if the construction technique is the same.

(a)

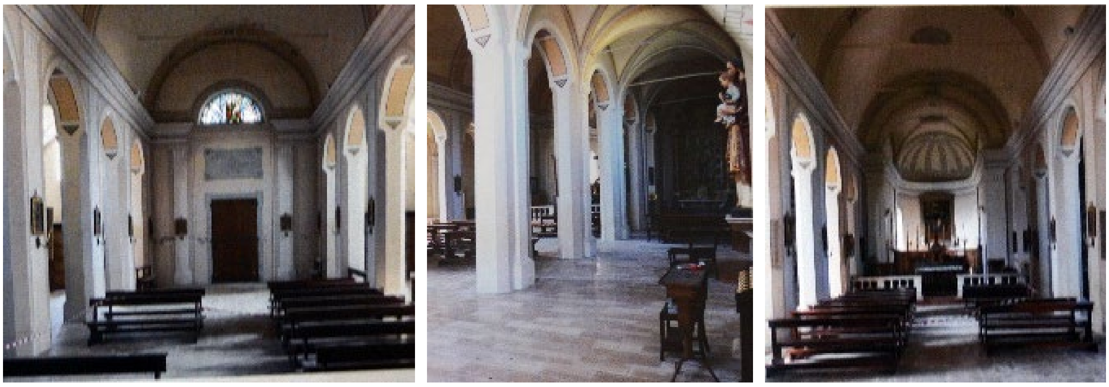
(b)


(c)


(d)


(e)


(f)



(g)

(h)

(i)

(l)

(m)

(n)

Figure 4- 9 The churches of the sample (Superintendence of Brescia 2018): **a.** San Lorenzo Diacono e Martire in Quingentole; **b.** Annunciazione della Beata Vergine Maria in Revere; **c.** Sant'Andrea Apostolo in Sarginesco; **d.** San Bartolomeo Apostolo in Quistello; **e.** San Giovanni Battista in San Giovanni del Dosso; **f.** San Giovanni Battista in Borgofranco sul Po; **g.** Santa Cecilia Vergine e Martire in Lobiola; **h.** San Giacomo e Mariano Martire in Villa Garibaldi; **i.** San Giacomo Maggiore Apostolo in Bonizzo; **l.** Assunzione della Beata Vergine Maria in Ostiglia; **m.** San Giacomo Maggiore Apostolo in San Giacomo Po; **n.** Assunzione della Beata Vergine Maria Felonica.

In some cases, groins interrupt the geometry of the barrel vault giving space to the lunettes of the windows. Moreover, the vault configuration frequently appears characterized by a series of consecutive vaults connected through arches. Such solution provides a rhythm to the nave and corresponds to the partitions on the wall between the naves (Figure 4- 9i). This wall is constituted by a series of arches and pillars or pilasters with an upper entablature, composed of a projecting cornice, a frieze, and a lintel. The entablature separates the lower order from the upper one. In some cases, the second order hosts openings of different shapes that give light to the principal nave. In other cases, the system of vaults is in direct contact with the entablature.

The use of timber is limited to the roof structure: in some cases, in the form of trusses, in others, in the form of simple rafters, without bottom chord.

Additional information on the church sample may be found in Annex II.

Some of these constitutive characteristics are common to other churches subjected to the 2012 Pianura Padana Emiliana earthquake presented in other studies (e.g. Sorrentino et al. 2014, Milani and Valente 2015, Valente et al. 2017, Valente and Milani 2018b).

4.2.2 THE OCCURRED DAMAGE

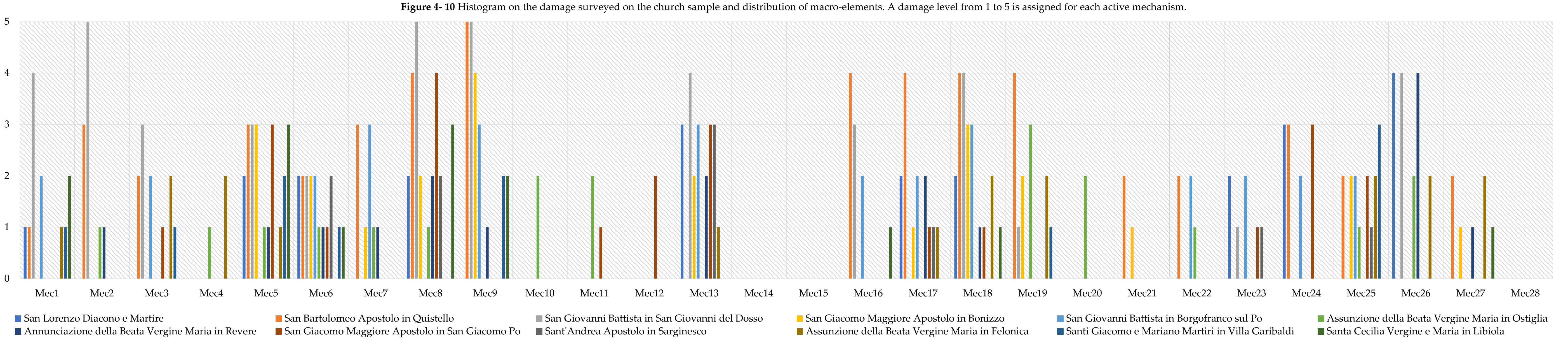
Figure 4- 10 reports the damage surveyed in the church sample after the two major shocks of the 2012 Pianura Padana Emiliana earthquake, distributed between the possible 28 mechanisms. The sources for the reported damage are the damage survey forms filled after the shock occurred on May 29. The surveyed damage is the result of the damage accumulation after the two major shocks.

Some specifications on the occurred damage are reported in the following:

- *The façade* suffered both in-plane and out-of-plane mechanisms. The former is mainly due to the presence of several openings; while the latter has been probably facilitated by the absence of longitudinal tie rods that could retain the overturning motion. The pilasters, typical of the baroque façade, seem to have contributed to contain the damage providing additional stiffness to the façade wall;
- *The façade gable* was one of the most damaged macro-elements, being highly overhanging from the roof level, a typical characteristic for most of the churches in the sample;
- *The embellishing volutes at the side of the gable*, even though not structural, got in some cases damaged due to the absence of retaining systems;
- *The pinnacles* are largely present in the church sample and they are distinguishing features in the façade configuration; they were often subjected to collapse due to their overhanging position in the façade. They are not a structural element but their presence and condition of vulnerability, which may have more general consequences, make such architectural element worth considering;
- *Brick walls* have shown a good monolithic behavior in all the cases where the state of conservation of mortar did not show degradation;
- *Triumphal arches* reported some damage but the presence of lateral walls at each side improved their structural response;

- *Vaults* were severely damaged, in particular the vault adjacent to the façade. This outcome may be associated to the extensive use of thin brick vaults. The arches between the vaults have demonstrated to be capable of isolating the damage, limiting its extension;
- *Colonnades* present shear cracks due to the longitudinal response of the structural system: the entablature develops a typical shear crack pattern;
- *Roofs (trusses, simple rafters*) were more influencing in the transversal response that in the longitudinal one. The box-like behavior was not significant. Some pounding effects from roof elements were observed in the masonry, concentrated at the connection areas.
- *Bell towers,* among the case examined, showed less damage compared to what generally occurred in other areas (e.g. Central Italy). This macro-element is frequently found isolated or structurally independent. Only 5 over 12 churches reported a low level of damage in the bell tower macro-element (corresponding to mechanism 27 in the abacus) and none in the belfry (mechanism 28). This is an interesting aspect if compared with what occurred in Central Italy, where the belfries are realized with a different material with respect to the rest of the bell tower, or with a different material pattern.

Figure 4- 10 Histogram on the damage surveyed on the church sample and distribution of macro-elements. A damage level from 1 to 5 is assigned for each active mechanism.



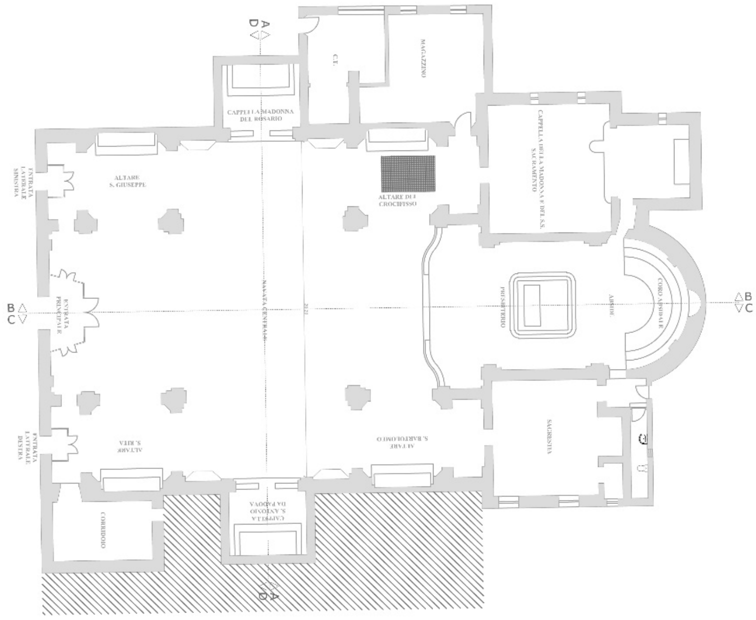
4.3 The case study: San Bartolomeo Apostolo in Quistello (MN)

The church of San Bartolomeo Apostolo in Quistello (MN) is taken here as a case study because it shows some of the characteristics identified in the churches of OltrePo. The church of San Bartolomeo Apostolo is located in the urban centre of Quistello, 180 m far from the Secchia river and 1 km from the Sabionello canal (Figure 4- 11).

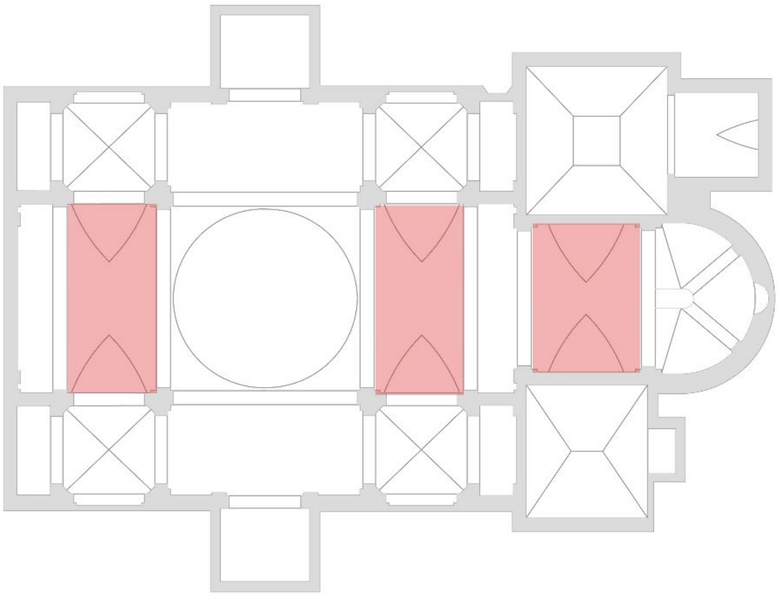
Figure 4- 11 Localization of San Bartolomeo Apostolo church in the urban centre of Quistello.

San Bartolomeo Apostolo was built between the 1730 and 1745 in replacement of an old church, located in a suburban area, according to the project elaborated by Arch. Giovanni Maria Barsotto. The church façade, with a brick fabric covered with plaster, characterizes the street scenery with its simplicity and typical features of the baroque churches of the OltrePo. The building has a canonic orientation with the apse on the east. In this case, the bell tower is independent from the rest of the church. Figure 4- 12 shows the plan of San Bartolomeo Apostolo composed by three naves, a presbytery, a semicircular apse, and annexed volumes.

The area highlighted in red in Figure 4- 12 is covered with barrel vaults made of thin brick layer; except for the central area covered with a dome, made of reeds ('camorcanna'), supported by four pendentives on top of the four composite pillars.



(a)



(b)

Figure 4- 12 Plan of San Bartolomeo Apostolo church: **a.** plan at the entrance level; **b.** plan at the vault level. The red area is covered with barrel vaults of thin brick layer.

The interior of the principal nave is marked by a thick cornice that turns around the church and creates a partition between the vaults, in the second order with the windows, and the first order rhythmed by pillars and arches that give access to the lateral naves (Figure 4- 13).

Figure 4- 13 Longitudinal section of the church.

The material mainly used is brick, as it is largely employed in the building stock of the area. Walls and vaults are built with such material, the latter are thin, with bricks laid flat. Thin brick vaults were common in the 18th century in this area and characterize local baroque churches. Figure 4- 13 clearly shows the rhythm that arches and vaults create in the church composition: a network of arches that sign the passage between one vault and another. The arches are composed of two wythes of bricks with empty segments that allow the intersection with the vaults.

The exterior of the church is simple. Only the façade is covered with plaster (Figure 4- 14), while the lateral walls show the pattern of the brickwork (Figure 4- 15).

Figure 4- 14 Façade of the church (Anzillotti and Fuentes 2018).

Figure 4- 14 shows the façade with three entrances, reflecting the same partition of the interior of the church. Above the lateral entrances, two niches reduce the section of the façade wall. Some pilasters regulate the design of the façade. Two cornices define the composition of the façade: a projecting cornice and an entablature separate the first from the second order, while another cornice divides the second order from the tympanum. At the second order, a pair of simplified volutes harmonizes the difference of dimension between the two orders characterizing the shape of the façade. A large central opening at the second order level has been infilled. On the top of the gable, a metal sphere and a marble base support the cross.

(a)

(a)

Figure 4- 15 Elevations and openings of the church (Superintendence of Brescia 2018): a. South elevation; b. Apse.

4.3.1 THE POST-2012 EARTHQUAKE DAMAGE

Figure 4- 16 shows the main structural units of San Bartolomeo Apostolo; out of all the possible macro-elements, only a few are present: façade, lateral walls, bell tower, apse, lateral chapels, triumphal arch, and vaults.

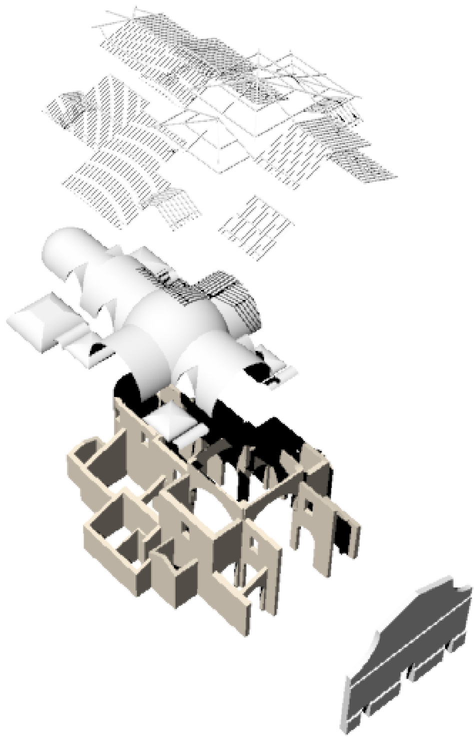
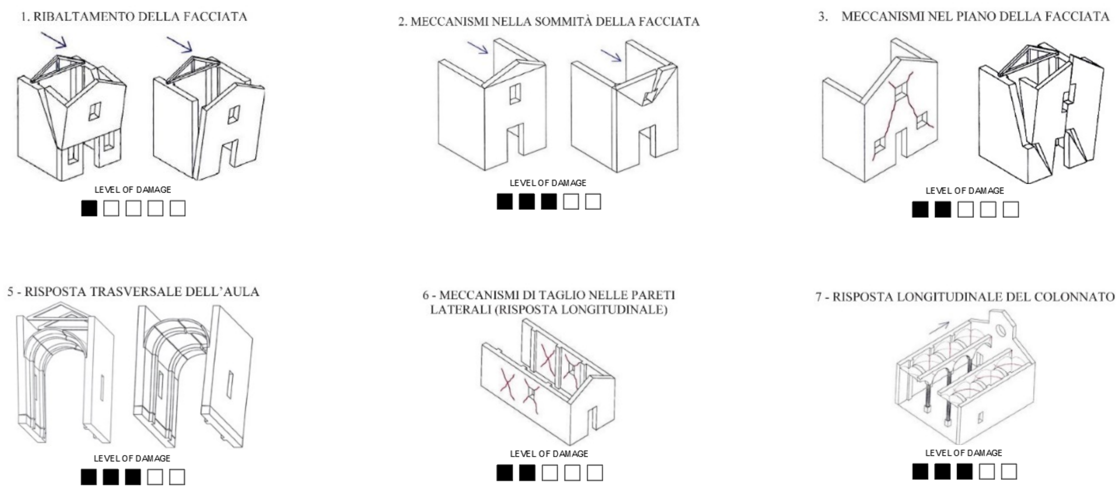


Figure 4- 16 Structural components of San Bartolomeo church. (Model credits: Anzillotti and Fuentes).

After the shock of May 29, the church was surveyed, Figure 4- 17 shows the mechanisms that developed. The damage is mostly concentrated on the façade, the system of vaults, and the colonnade. This is an interesting aspect if considered in relation with the configuration pointed out in the description of the case study (Section 4.3) and, more in general, with the construction characteristics of this geographical area (Section 4.2.1).



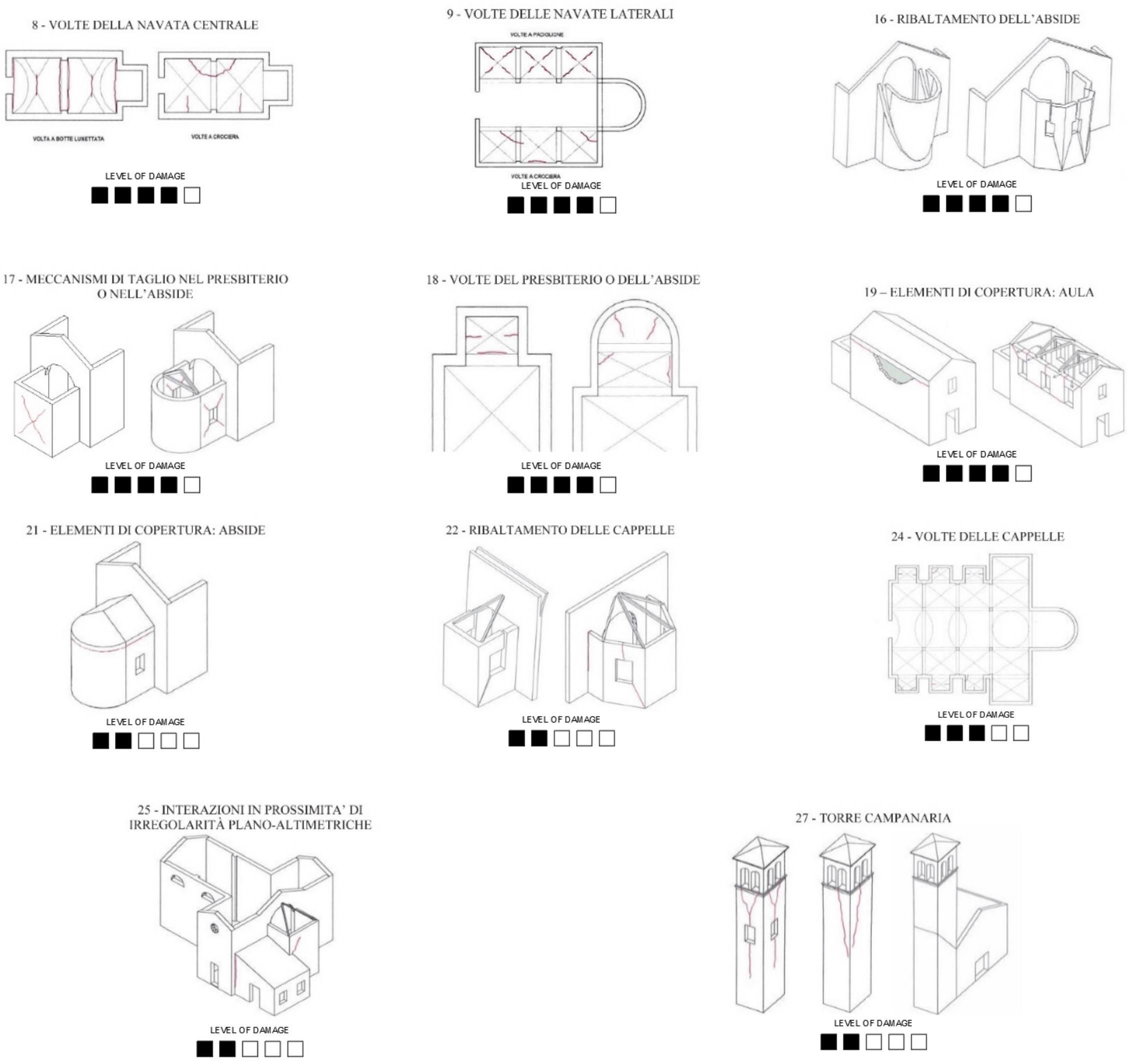
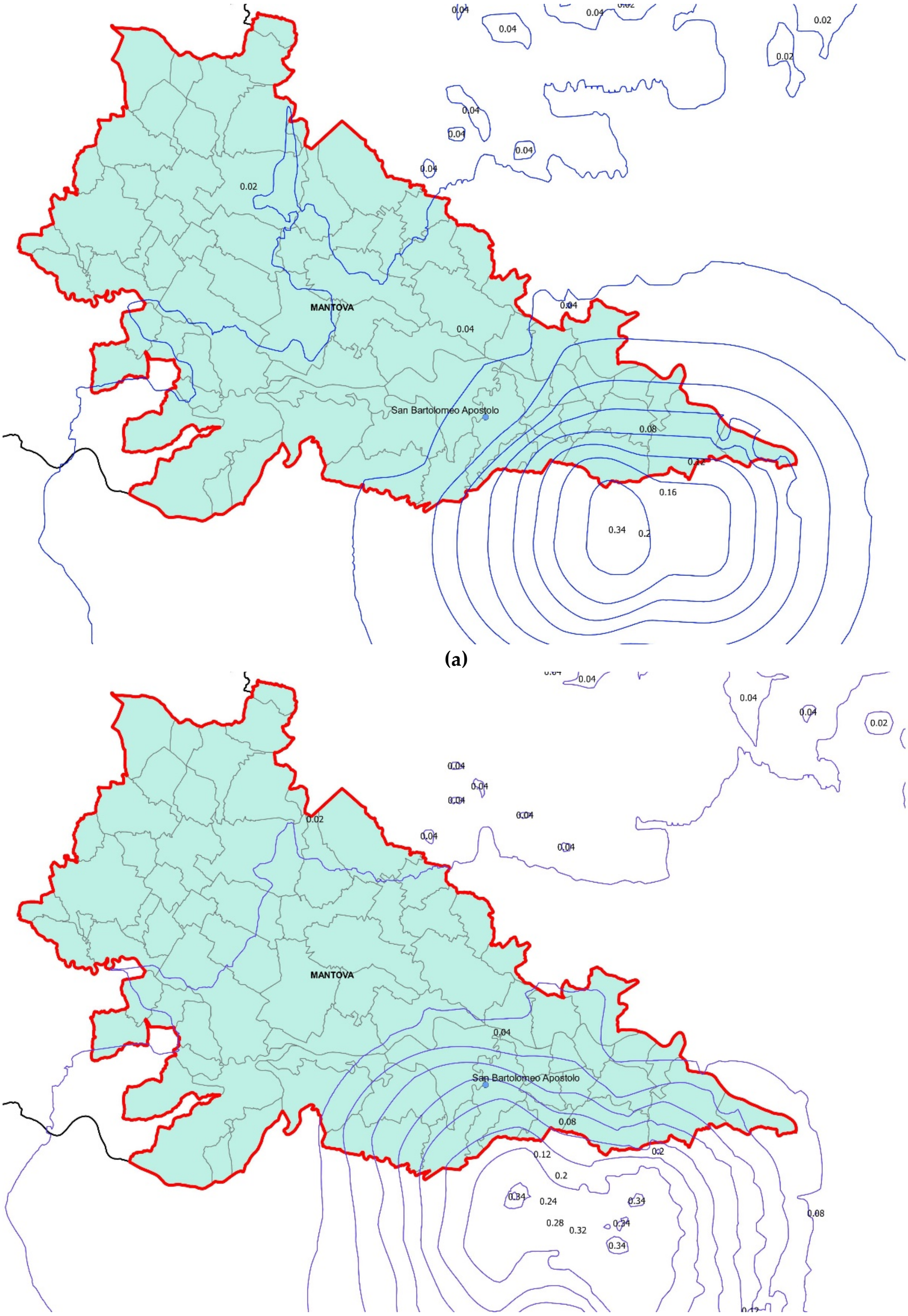


Figure 4- 17 The occurred damage, distinguished per mechanism, after the two major shocks on May 2012. Mechanism 1: overturning of the façade; Mechanism 2: mechanisms of the upper part of the façade; Mechanism 3: in-plane mechanisms of the façade; Mechanism 5: longitudinal response of the lateral wall, in plane; Mechanism 6: Shear cracks in the walls; Mechanism 7: longitudinal response of the colonnade; Mechanism 8: crack in the vaults of the principal nave; Mechanism 9: cracks in the vaults of the lateral naves; Mechanism 16: Overturning of the apse; Mechanism 17: shear cracks in the presbytery or apse; Mechanism 18: cracks in the vaults of the apse; Mechanism 19: roof elements in the principal nave; Mechanism 21: roof element in the apse; Mechanism 22: overturning of the chapels; Mechanism 24: crack in the chapels; Mechanism 25: Crack in the part of interaction with irregularities; Mechanism 27: Cracks in the bell tower.

Such damage can be explained not only in relation with the construction characteristics of the church, that is the vulnerability associated with the building, but also at territorial level considering the PGA reached in the area with the two seismic events (Figure 4- 18). The two strong shocks hit the church in a short time lapse. According to the map, generated overlaying the position of the church with the PGA isocurves, in the absence of onsite registrations, San Bartolomeo stays in the 0.02 g area for the first shock (Figure 4- 18a) and 0.08 g for the second (Figure 4- 18b). The relatively short time between one seismic event and the other, which prevented repairing or just provisional interventions, and the increase values of PGA in the two events contribute to explain the progression of damage, as observed in the case of the vault adjacent to the façade.



(b)

Figure 4- 18 The church and the PGA isocurve: a. on May 20; b. on May 29.

The activation of mechanism 2, overturning of the gable, caused the collapse of the marble element, located on the top of the façade supporting the cross, towards the interior of the church. Such collapse caused the failure of part of the roof immediately behind the façade and, consequently, the vault adjacent to the façade was partially damaged (Figure 4- 19a). The second shock worsened the damage causing the complete collapse of that vault (Figure 4- 19b) together with other vaults covering other areas of the church (Figure 4- 19c).

(a) **(b)** **(c)**

Figure 4- 19 Collapse of the vault of the church (Superintendence of Brescia 2018): **a.** the vault adjacent to the façade after the shock occurred on May 20; **b.** the same vault after the shock on May 29; **c.** collapse and damage of other vaults of the church.

After the second shock, also the organ and the internal frame of the first span of the church were severely damaged (Figure 4- 20).

Figure 4- 20 Collapse of the organ area behind the façade after the second shock (Superintendence of Brescia 2018).

Shear cracks appeared in the apse (Figure 4- 21a). The arches of the longitudinal walls, diaphragms between central and lateral naves, were damaged (Figure 4- 21b). Such cracks on the arches extended up to the vaults of the central nave, with the collapse of some parts (Figure 4- 21c).

(a) (b) (c)

Figure 4- 21 Other damage on the church (Superintendence of Brescia 2018): **a.** shear cracks in the apse; **b.** shear cracks in the arches of the colonnade; **c.** collapse of other vaults of the church and damage of the central dome.

4.4.2 NUMERICAL RESULTS

Here, the numerical results needed for the comprehension of the seismic vulnerability of the church are reported. The implemented model has a mesh made of 57144 tetrahedral elements. For the analyses an equivalent homogeneous isotropic material was chosen with a Young's modulus equal to 1500 KN/m^2 and a mass density of 2100 Kg/m^3 for the masonry walls, while 1500 KN/m^2 and 1800 Kg/m^3 for the vaults, reported in the Commentary of the Italian Building code (CMTI 2019). In the model with the roof beams, the rafters were modelled with a material with Young's modulus equal to 15000 KN/m^2. To simulate the post-elastic behavior of the material, the concrete damage plasticity (CDP) model was chosen (Lubliner et al. 1989, Lee and Fenves 1998). Other detailed information on the elaboration of the numerical model may be found in Anzillotti and Fuentes (2018).

4.4.2.1 MODAL ANALYSIS

In Anzillotti and Fuentes (2018), a numerical model was implemented with and without the roof rafters. Here the modal shape of the church configuration with the roof rafters are reported. In fact, from the comparison of the two model configurations, the contribution of the roof results significant in making the structure stiffer, reducing the values of displacement, with results that appear to better correspond to the observed damage.

The results of the performed modal analyses are shown in Figure 4- 22. Mode 1 has a participating mass ratio in the longitudinal direction of the church of 9.91%, a relatively low value. Indeed, motion is involving only the tympanum. This mode is significant if compared with what occurred. The first shock caused the collapse of the marble element supporting the cross in consequence of the displacement of the gable. Mode 2 shows the highest participating mass ratio with 43.55% in the transversal direction. Mode 3 shows a higher value of participating mass ratio in the longitudinal direction, 14.49% concerning once more the gable and the vault adjacent to the façade. Mode 6 symmetrically interests the lateral volutes of the façade, typical of the baroque configuration. To reach about the 70% of the total modal mass in longitudinal and transversal direction, 50 modes need to be analyzed. This condition is indicative of a response that builds up mainly with local contributions.

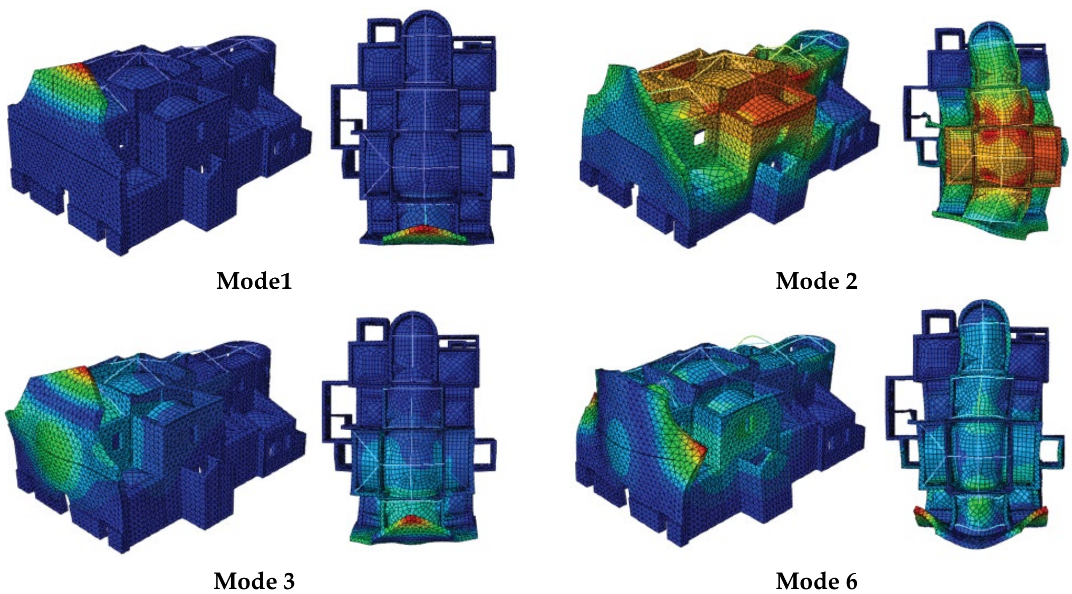


Figure 4- 22 Significant modal shapes.

4.4.2.2 Kinematic limit analysis

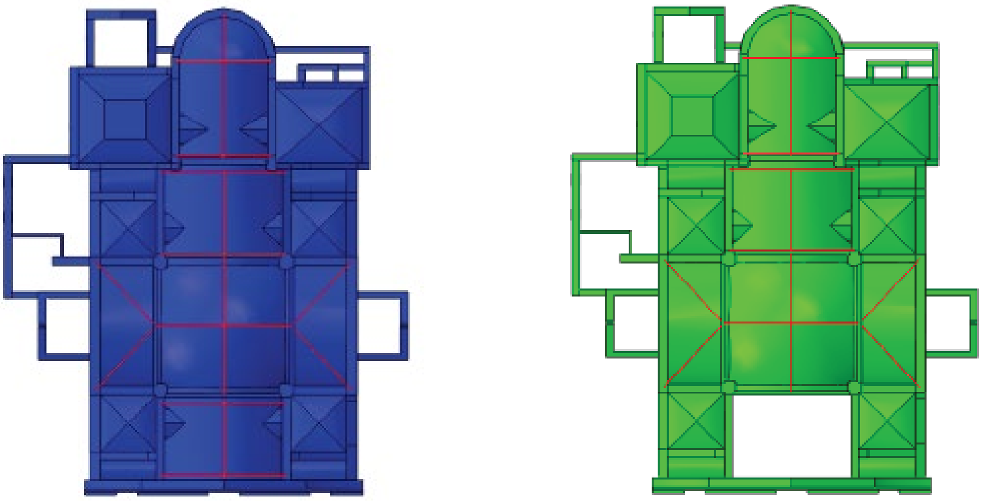
From the damage survey of the church, the level of damage for the mechanism of gable overturning was assessed as equal to 3 (Figure 4- 10). As remarked in Section 4.2.1, the churches with baroque façade typically present a soaring gable, usually without any retaining system to avoid overturning. In order to better understand the structural behavior of the gable for San Bartolomeo, representative of similar macro-elements in the baroque typology, a limit analysis with kinematic approach was performed (Anzillotti and Fuentes 2018). The analyses were performed assuming:

- Absence of retaining element at the top of the gable;
- The gable as a rigid body able to rotate around a hinge at the base of the macro-element.

From the performed analyses, the acceleration capacity resulted equal to 0.125 g. The Damage Limit State resulted not verified at the elevation of 15.2 m, where the mechanism is activated, and the acceleration demand was 0.179 g. The ultimate limit state, at the same elevation, was also not verified. At the church site the ground acceleration could be estimated as 0.08g, both on May 20 and 29, as the elaborated GIS map shows (Figure 4- 18). The results from the limit analysis confirmed the activation of the overturning mechanism of the gable that developed during the two shocks of May 2012.

4.4.2.3 Time History Analysis

Another type of analysis performed on the church is the time history analysis in order to understand the dynamic response of the church structure over time. This analysis was performed both in the linear and non-linear modality. The linear analysis cannot, indeed, describe the full response, but it has been deemed useful for defining some structural characteristics and verifying some assumptions in view of preparing a suitable simplified numerical model for the nonlinear analysis that requires a major computational effort. The analyses were performed on two global models, one that represent the complete church and another without the vault adjacent to the façade, in order to simulate a similar condition to what occurred with the shock of May, 29 (Figure 4- 23).



(a) **(b)**

Figure 4- 23 Model used for the linear time history analyses: **a.** complete structure; **b.** Structure without the vault adjacent to the façade.

The comparison of these two models is significant for understanding the contribution of the specificity of such typology, pointed out in section 4.2.1, concerning the frame constituted by arches and vaults. The principal nave of the baroque churches is constituted of a series of spans covered by barrel vaults and connected one to the other through arches. From the post-earthquake surveys of the churches, the vault adjacent to the façade was usually damaged. In some cases, the damage was limited to this portion and did not extend to the vault nearby. In the performed analyses, the accelerogram from the recording station of Poggio-Rusco was used and the East-West component of motion for May 29 was applied. It corresponds to the longitudinal direction that resulted the most critical from the modal analysis. The accelerogram was limited to the range 41 -47s (Figure 4- 24).

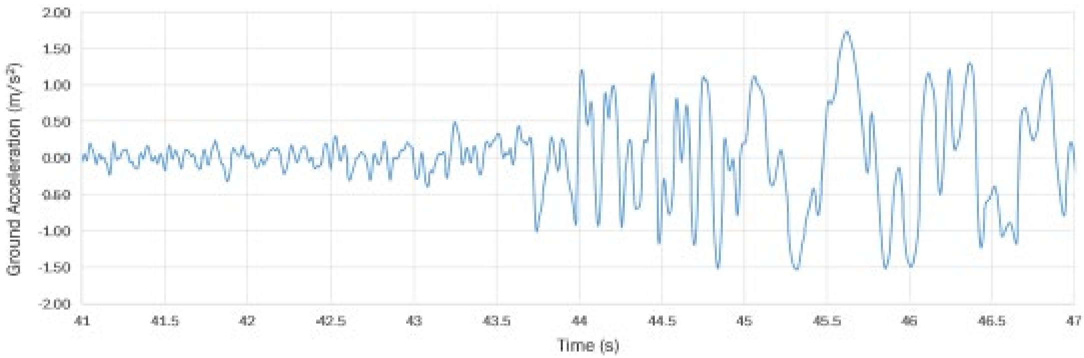


Figure 4- 24 Accelerogram East-West component of the shock occurred on May, 29.

First of all, the influence of the presence or absence of the vault adjacent to the façade was evaluated based on modal analysis. Table 4- 1 reports the periods of the modal shapes of the two model configurations, M1 and M2 respectively. The maximum difference between the periods of vibration of the two models is in the first mode that concerns the façade macro-element. The presence of the vault behind the façade provides stiffness to the structure. Indeed, the influence is local and does not involve the complete structure.

Table 4- 1 Comparison of the period between the complete model (M1) and the model without the vault (M2)

	M1			M2		
Mode	**% Participating mass ratio (Longitudinal)**	**% Participating mass ratio (Transversal)**	**Period**	**% Participating mass ratio (Longitudinal)**	**% Participating mass ratio (Transversal)**	**Period**
1	8.56	0.0001	0.354 s	5.74	0.0001	0.440 s
2	0.00	46.11	0.294 s	0.00	41.47	0.307 s
3	25.51	0.01	0.240 s	26.38	0.00	0.240 s
4	0.01	0.78	0.215 s	0.15	1.04	0.237 s
5	0.30	0.48	0.209 s	0.13	0.27	0.219 s
6	6.96	0.41	0.191 s	3.54	0.01	0.209 s
7	0.00	3.66	0.180 s	4.27	1.13	0.188 s
8	1.37	1.92	0.175 s	0.85	7.34	0.181 s
9	0.10	0.01	0.167 s	0.34	0.23	0.174 s
10	6.39	0.16	0.161 s	0.01	0.32	0.169 s
11	6.01	0.03	0.155 s	1.12	5.91	0.164 s
12	0.17	4.39	0.153 s	10.93	0.36	0.161 s

In order to support this last statement, the displacement of specifically identified points of the structure was checked(Figure 4- 25): the highest point of the façade (1), the back side of the façade close to the intersection with the vault adjacent to the façade (2), and the points at the top of the other vaults of the church (3,4,5,6,7).

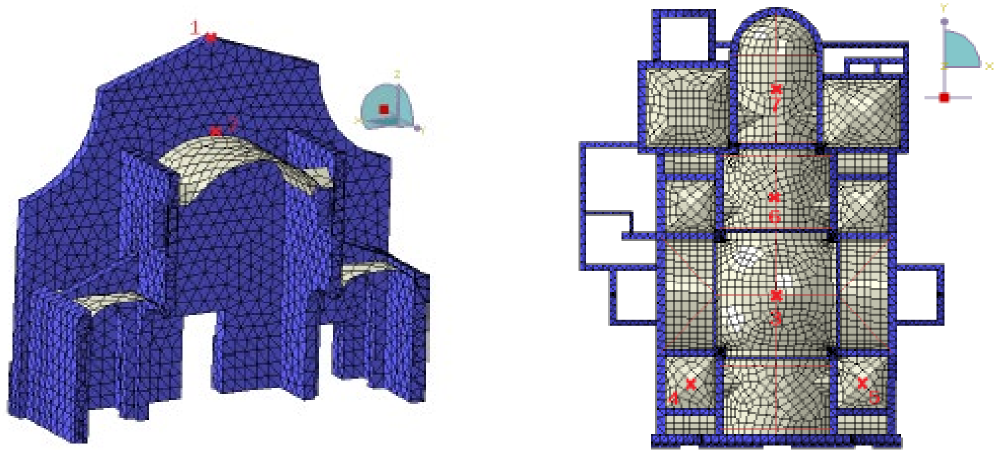
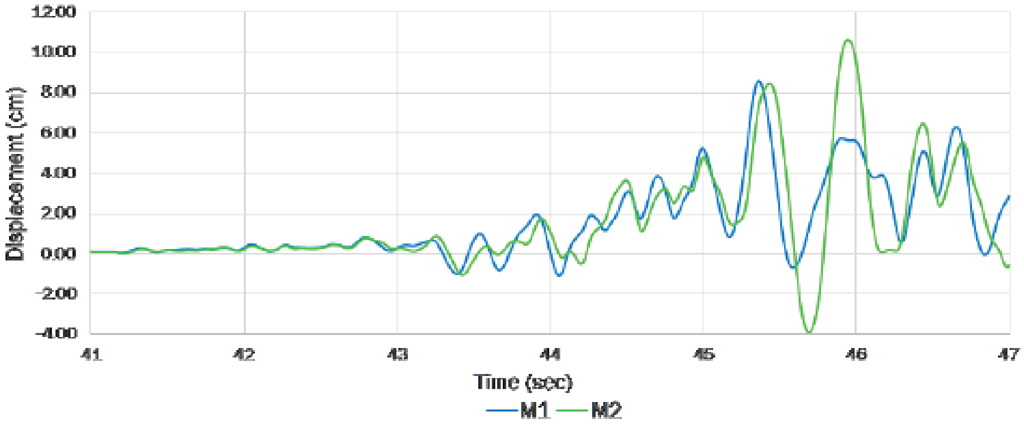


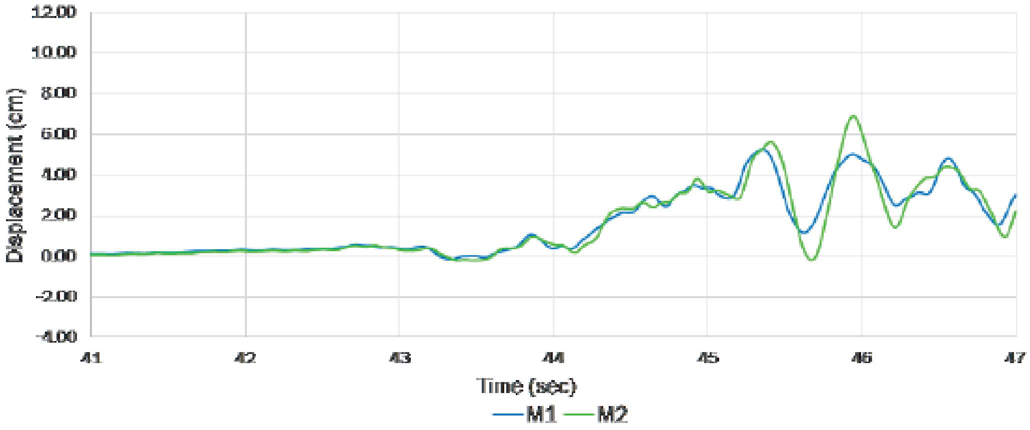
Figure 4- 25 Position of the points (1,2,3,4,5,6,7) whose displacement was investigated (Anzillotti and Fuentes 2018).

Figure 4- 29a shows two displacement curves, one for the complete model, M1 and another for the model without the vault, M2 in the linear assumption. It results that point 1 has the maximum displacement (9 cm) at T=45.3 s in the first model, while in the second configuration it is reached at 46 s with a displacement of 11 cm.

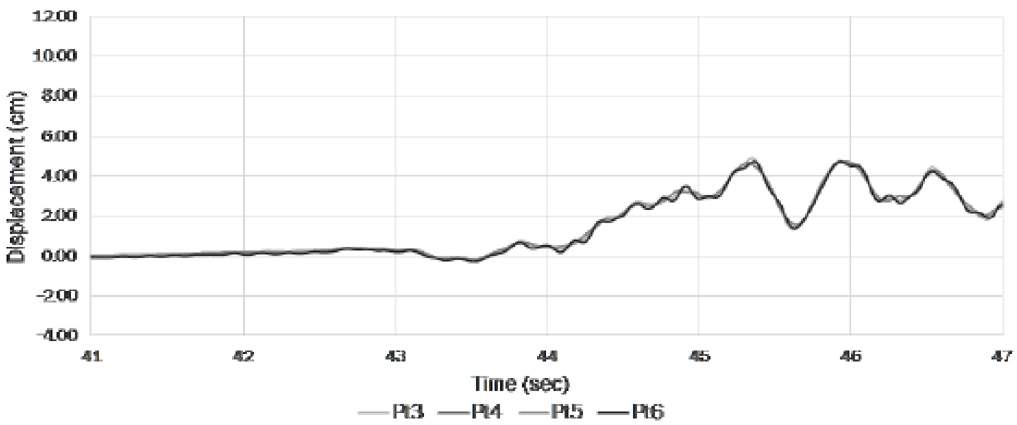
Figure 4- 29b plots the curves for the point taken in the back of the façade at the intersection with the vault showing the same trend but with a reduced amplitude in the complete model. On the contrary, Figure 4- 29c shows the points taken in the distinct vaults in the church that do not show any significant variations between one model and the other. In this regard, the hypothesis was formulated that the frame structure of arches contributes in isolating the effect produced by the collapse of the vault.



(a)



(b)



(c)

Figure 4- 26 Plot of the displacement of the identified points (1,2,3,4,5,6,7): **a**. Point at the top of the façade; **b.** Point the back side of the façade at the level of intersection with the vault; **c.** Points taken on the vaults in the complete model (M1).

Moreover, from the performed linear time history analysis, the stresses in the church were analyzed considering only the seconds up to the achievement of the elastic limit. The maximum principal stresses, in the same range considered before (from 41 to 47 s), show that the church structure reaches the elastic limit in the façade and in the vault adjacent to the façade more or less at 43 s. The comparison of the stresses for the two church configurations at T=43 s is shown in Figure 4- 27.

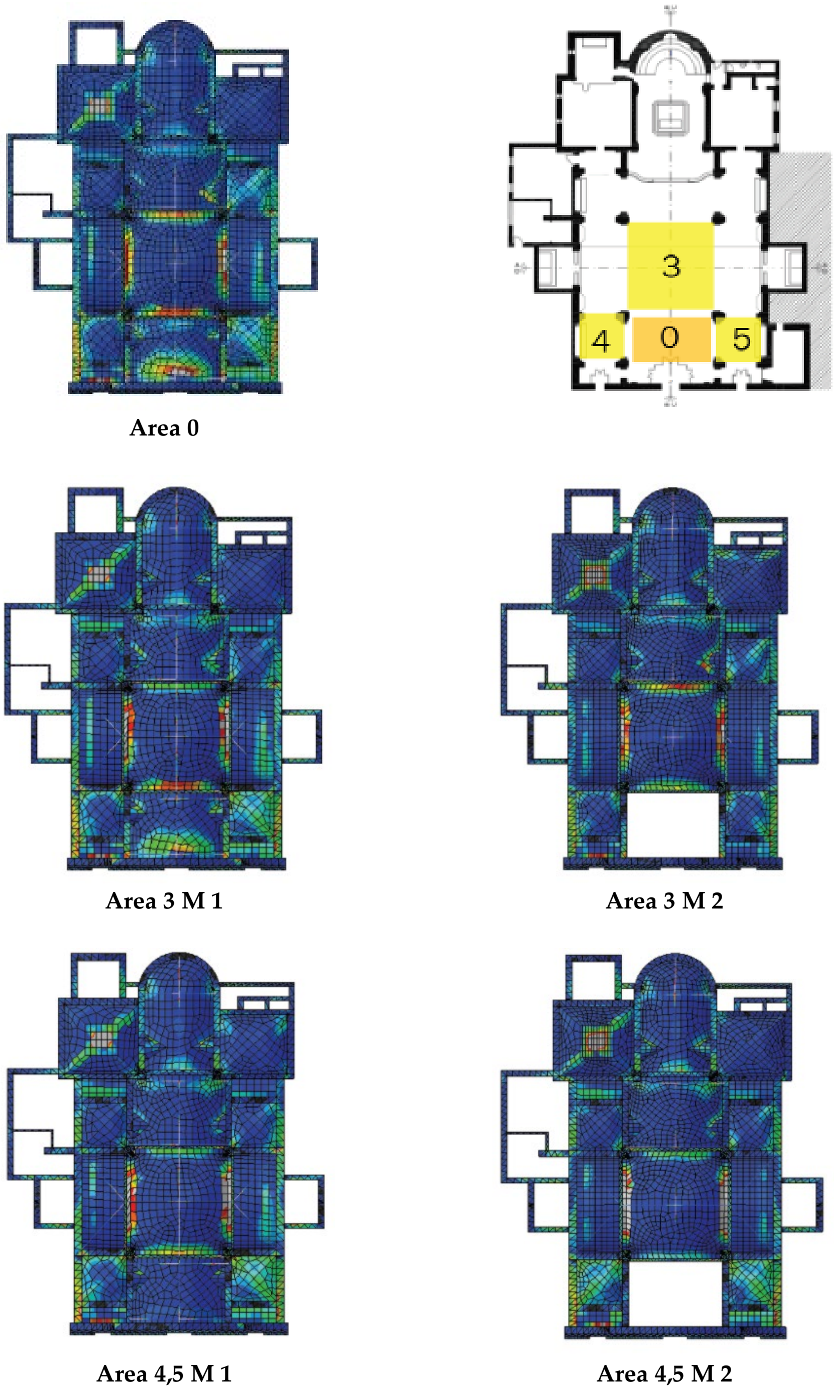


Figure 4- 27 The comparison of stresses at 43 s of the two church configurations: with vault (M1), without vault (M2). Other plots during the range between 41 and 43.5 s can be consulted in Anzillotti and Fuentes (2018, p.137-140).

In summary, the linear time history analysis has demonstrated that the collapse of the vault adjacent to the façade has a local influence in the structural behavior, visible both observing the displacements (Figure 4- 26) and the comparison of the stresses between the two configurations (Figure 4- 27). Moreover, applying the accelerogram in the longitudinal direction the structure reaches at 43 s the maximum stress capacity in those areas resulted from the analysis as the most subjected to stresses: the façade and the vault adjacent to the façade.

A non-linear time history analysis was then performed applying again the recorded accelerogram. Given the size of the structure, this type of analysis was restricted just to the first span of the church (Figure 4- 28). In this way, the structural behavior of the façade and the barrel vault, which totally collapsed during the shock on May 29, was studied. Moreover, such decision was taken also considering the results of the previous analyses that show the most critical area in terms of displacement and stresses to be the façade and the vault adjacent to the façade.

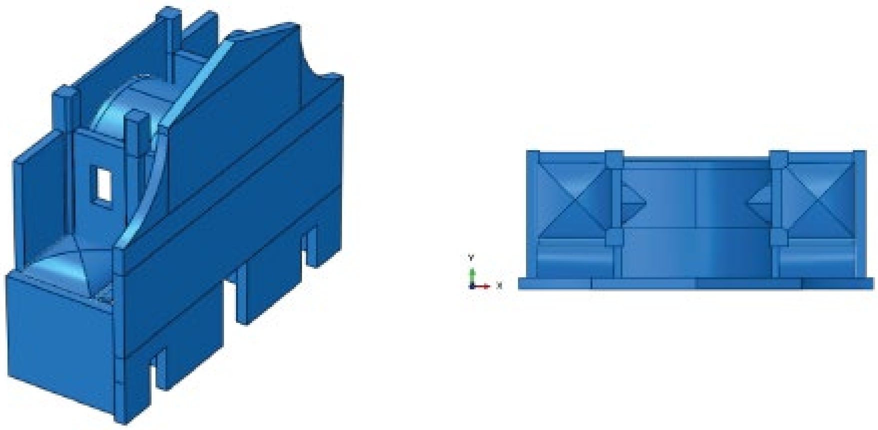
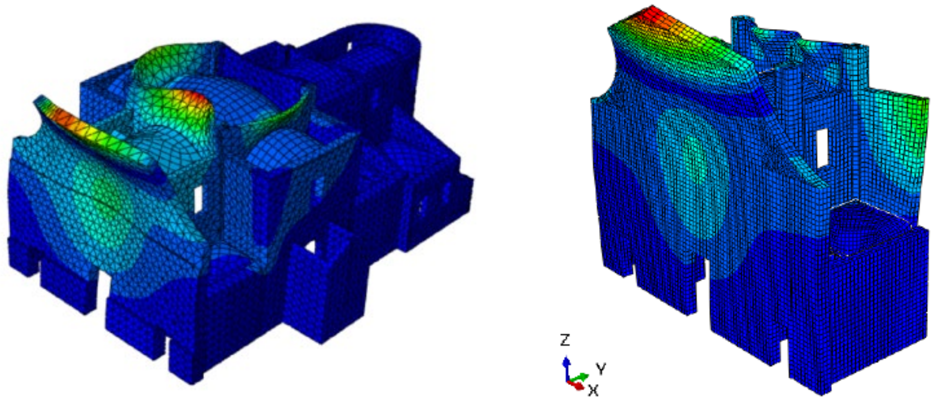


Figure 4- 28 Portion of the church that was used for the non-linear time history analysis.

The analysis was performed considering again a restricted portion of the seismic shock sequence. Such significant range was calculated, following Trifunac and Brady (1975) and it was estimated being restricted to 5 seconds, from 43 to 48 s. First of all, in order to consider it reliable, the partial model modal analyses were performed for this model and the period that interests the gable was compared with that of the full structure. The period of the complete model was equal to 0.38 s, while that of the partial model was equal to 0.33 (Figure 4- 29). The similarity of the periods confirmed the viability of the approximation that would result in considering only the first portion of the model.



(a) **(b)**

Figure 4- 29 The comparison of the modal shapes: **a.** the global model; **b.** the partial one.

The non-linear time history analysis was performed recurring to a CDP material (see section 3.3.3). Two significant points were selected in order to understand the structural behavior of the church during the seismic shock: the top of the façade (1) and the point of connection between the vault and the façade (2). Figure 4- 30 shows the displacement trend of the two points between 43 and 48 s. Point 1 and Point 2 are characterized by a different oscillation. Up to 44 s, the displacement of the gable has a wide amplitude. After 44 s, the structural behavior changes and point 2 registers a larger displacement. The façade after the separation from the vault behind it moves more as a rigid block, that includes the top that previously oscillated with larger relative displacements.

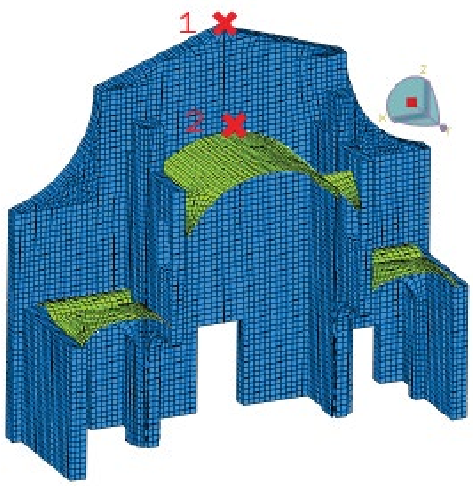


Figure 4- 30 The displacement of two points of the façade: the top of the façade (1), the point of connection between the vault and the façade (2).

Observing the stress distribution, some considerations can be added:

- The stress distribution moves from the bottom of the façade toward the gable at 44.5 s, with a stress concentration in the point of connection between the vault and the façade (Figure 4- 31a);
- at 45 s the stresses are concentrated in the point of connection between the façade and the longitudinal walls of the principal nave and in the vaults of the lateral ones (Figure 4- 31b);
- at 45.4 s the stresses in the façade design a ‘v’ shape typical of the mechanism of the gable (Mec. 3)(Figure 4- 31c);

  at 46 s the stresses reach the top of the gable (Figure 4- 31d).

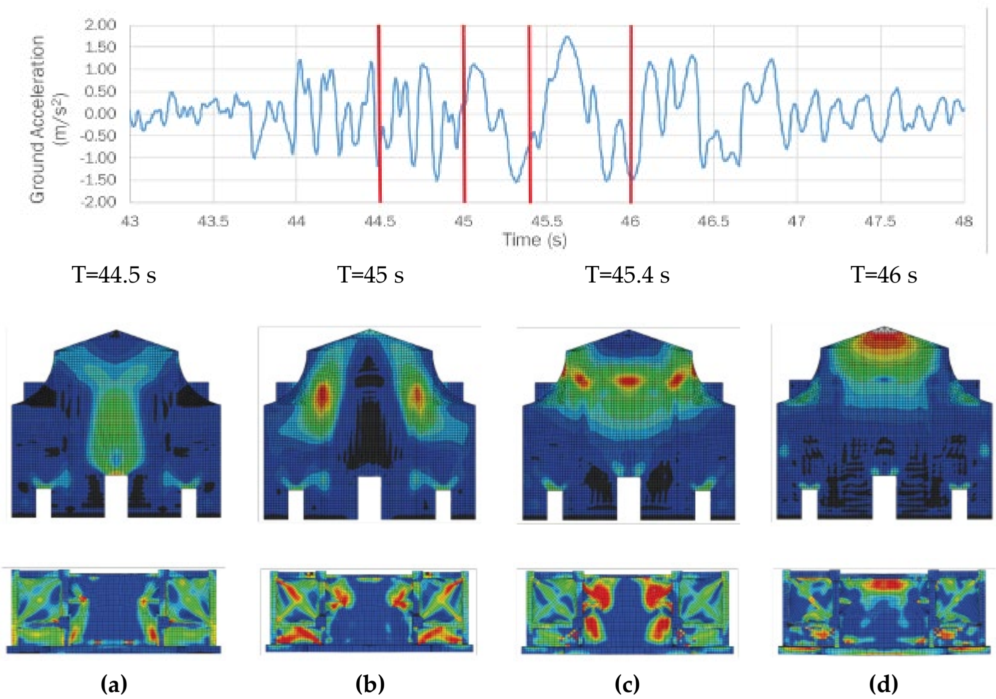


Figure 4- 31 Stresses in significant seconds.

The results show a critical zone in the area of contact between the façade and the vault. Observing the damage sequence for specific time intervals, the damage develops in such part (Figure 4- 32). Such information together with what observed from Figure 4- 31 provides an explanation of why the gable did not collapse after the shock of May 29 even if already compromised after the first shock on May 20. When the connection of the façade to the lateral walls degraded and the façade as a whole started to detach from the nave walls, a passage from a gable mechanism that was developing over an initially stiff base to a full façade mechanism took place. The earthquake luckily ended before the total collapse of the façade, leaving serious but recoverable damage.

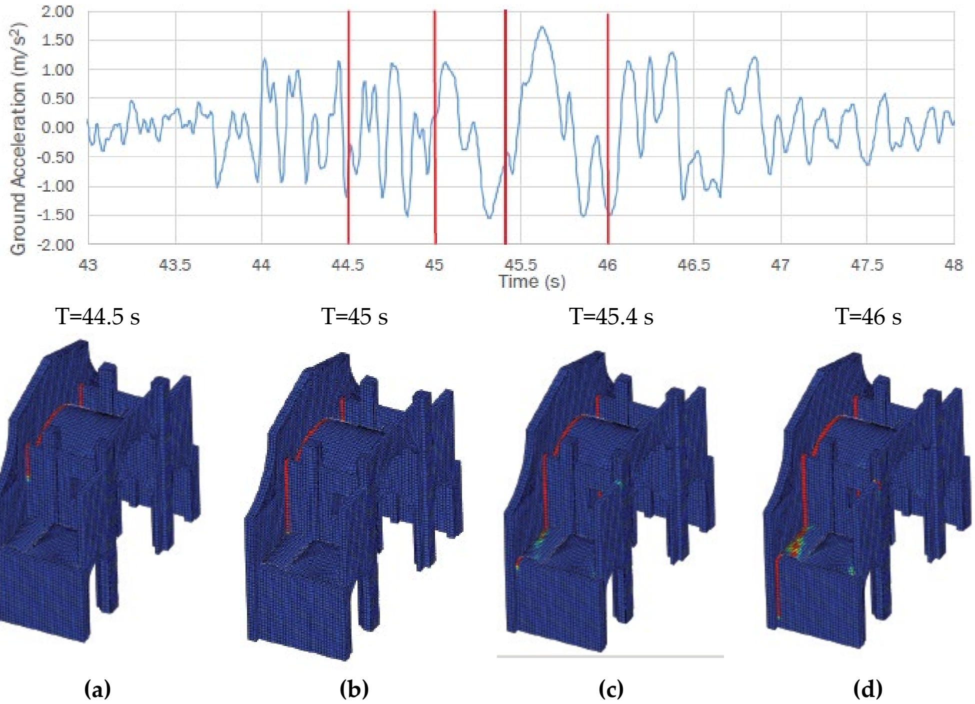


Figure 4- 32 Open cracks of the church for selected seconds.

4.5 Territorial aspects from the sample

After analyzing the damage produced by the Pianura Padana earthquake of 2012 on a sample of churches and developing in depth a specific case study, it seems possible to sort out for these buildings some *territorial specificities*, summarized in the following.

In the churches examined, the state of conservation of the mortar was found good. When such condition is ascertained, the behavior of the brick walls usually results monolithic and no separation of the masonry leaves is detected. On the contrary, when this is not guaranteed partial brick collapses could happen. The state of conservation of the mortar is crucial for this geographical area. Indeed, surveyors reported for the area an extensive use of river sand for the mortar production, due to the location of the historical buildings close to the river. Such mortar composition sometimes showed phenomena of degradation.

The vault adjacent to the façade, built as a vertical stacked bond shell, confirmed to be a particularly vulnerable part of the building. At the same time, this particular element, being in contact with the façade, suffered damage also due to the vibration and pounding effect of the façade wall when this last became not sufficiently retained by the longitudinal walls of the nave or longitudinal tie rods.

A typical aspect of the baroque configuration is the system of arches that separates each vault confining it in a sort of three-dimensional frame. This feature reduces the vulnerability of the horizontal coverings, isolating the damage in case of earthquake.

In similar churches of this geographical area, thicker masonry ribs were found in the extrados of the thin brick vaults to make them stiffer.

The façade macro-element has frequently shown the activation of the full overturning mechanism due to the absence of longitudinal retaining systems.

In the baroque typology as implemented in this area, the presence of a strongly soaring gable becomes a specific source of vulnerability.

Depending on the history of construction, the bell tower sometimes is annexed to the triumphal arch. Such condition motivated the increment of the vulnerability for the triumphal arch that suffered damage when the bell tower was loaded by lateral forces.

On the contrary, when the arch, as in most cases, was connected with a wall per side (the walls of the lateral nave or chapels), its vulnerability appeared reduced by exploiting collaboration in the response.

CHAPTER 5

AN EXAMPLE OF CHURCH-FORTRESS CONFIGURATION IN EASTERN SICILY: INTERVENTIONS AND TYPOLOGICAL VULNERABILITY

This section investigates a specific case, La Badiazza, a church-fortress located in Messina. This is an example of configuration similar to other churches that can be found in Sicily and that are the result of the construction practice of this area and its history of dominations. In the specific case, the church is in Eastern Sicily, an area rather different for seismicity from Central Italy, being characterized by few but strong earthquakes (Giuffré 1993). It is sufficient to remember the Stretto di Messina earthquake of 1908. Since that time, no strong earthquake has occurred, but the approaches to interventions over the years have influenced the current state of the churches.

This chapter investigates the structural behavior of a church similar to a fortress and with the characteristic layout composed of two main volumes, the square presbytery and the tripartite nave space, features common among the churches of the 11th and 12th century in this area and a result of its history. This church example is worth studying because it differs from those treated in Central Italy and Lombardy and testifies the importance to develop a methodology capable of including the territorial specificities. Assessing the seismic vulnerability of such church configuration with the list of the 28-mechanisms (MiBAC 2011) would be too reductive in explaining the real structural response of this and other similar churches. The case study is *Santa Maria della Valle*, locally called "La Badiazza", in Messina. This eastern part of Sicily is well known for its seismicity; in particular, an event worth noting for its massive impact is the Messina-Reggio Calabria earthquake, 1908, which caused thousands of victims. Observing the seismic history of the area (Figure 5- 1), such strong event is immediately identifiable, as its intensity value in the Mercalli scale is the highest ever. The church underwent interventions and changes of configuration at different times. Understanding the influence of such changes on the structural response is significant for the comprehension of the seismic vulnerability of this church typology. Among various interventions and changes, the influence of the substitution of a masonry column in the colonnade with one with a concrete core is investigated (Section 5.2.2). Such intervention is the result of a restoration approach typical of the 1980s.

The contribution reported here derives from the work by Fleres (2017), for which supervising support was provided. It is also the source of figures, 5 – 1 to 5 – 17, with some minor modifications.

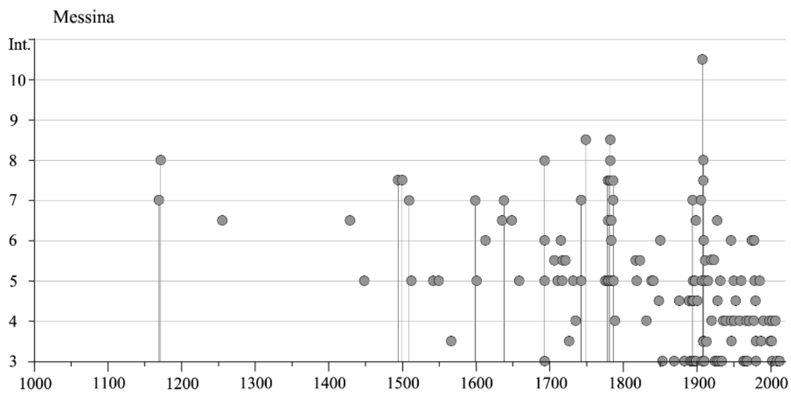


Figure 5- 1 The histogram represents the year of occurrence of the earthquake as abscissa and the intensity value of Mercalli Cancani - Sieberg Scale as ordinate. Source of data (Locati et al. 2019).

5.1 Notions on the Arabic Norman Architecture in Eastern Sicily

In the definition of expressive characters in the architectural features, the builders of Sicily did not act independently from other areas of Southern Italy under Norman influence. In Sicily, according to E. Calandra, the religious buildings, built in the 12th century, followed two principal artistic movements that can be distinguished depending on the area of development (Ciotta 1992). In Western Sicily, in the area called *Val di Mazara*, the Islamic culture was stronger, and it has been translated in stereometric and prismatic volumes, exalted by walls made of small bricks, dressed, without soaring details and with openings marked by pointed arches. In Eastern Sicily, in the area of *Val Demone*, the builders, still anchored to the byzantine tradition, recurred to chromatic effects realized with red bricks and layers of white mortar, dark and light stones and columns connected with arches. F. Basile, sharing Calandra's theory, developed a careful study on the churches of Val Demone (San Filippo di Fragalà in Frazzanò, Santa Maria di Mili, San Pietro di Itala, SS. Pietro di Casalvecchio Siculo) and retraced some peculiar aspects of the buildings in this period (Ciotta 1992). They are summarized here:

- *the recurrent use of distinct colors for the external walls.* Such effects are obtained with stones and mortar of different types;
- *the planimetric configuration.* This is characterized by a transept with small arms or without them;
- *the simple and slender volumes* that create the composition;
- *the development of a constructive practice* capable to solve the static problem by correctly connecting the dome and its impost, often in a rectangular space;
- *the use of the arch* that anticipates the gothic shape; it characterized in particular the second part of the 12th century.

G. di Stefano subdivided the Norman buildings in four periods (Ciotta 1992):

- *Della conquista e della contea* (1061-1130) (*conquest and county foundation*). It is considered an experimental period for the features that were confirmed later. The common planimetric development is the latin cross. From a construction perspective, there is an extensive use of the pointed arch imposed on tall pillars, of small domes that insist on niches with or without tambour, of columns that are relegated to corners of the walls, and a limited use of groin vaults. Finally, at the exterior a peculiar attention to the decoration of the apses is paid. All these aspects recalled the Islamic culture;

- *Ruggero II period* (1130-1154). The richness of byzantine and muslim cultures fascinated the king Ruggero II. He promoted the construction of several buildings with constructive and decorative features already experimented in the previous period but enriched with the influences coming from the new exchanges with North Africa. Ruggero II ordered the construction of the cathedrals of Cefalù and Messina that symbolizes this period.

- *Guglielmo I period* (1154 – 1166). During the short kingdom of Guglielmo I there was a scarce realization of buildings except for the church of San Cataldo (Ciotta 1992p.122) and the Zisa palace (first phase) in Palermo;

- *Guglielmo II period* (1166-1189) characterized by compact plan and cubic volumes. For their planimetric development and configuration, the churches of Annunciata dei Catalani in Messina and of Santo Spirito in Palermo are examples of this period.

5.2 LA BADIAZZA

The church of Santa Maria della Valle is commonly known, among the inhabitants, as La Badiazza, due to the name of a stream that flows close by.
La Badiazza is located in the upper part of the narrow valley outside the urban area of Messina and near the pass of the Peloritani mountains. It appears isolated, with a north-west/south-east orientation (Figure 5- 2), and stands out as a fortress in the natural landscape (Figure 5- 3).

Figure 5- 2 La Badiazza in the valley outside the urban area of Messina and near the pass of the Peloritani mountains.

(a) **(b)**

(c) **(d)**

Figure 5- 3 La Badiazza: **a.** north-eastern view; **b.** Principal façade (north-western view); **c.** View from above; **d.** North-eastern view (Photo rights R. Fleres).

From the exterior, some aspects that may be assumed as representative for the typology can be observed. The façade as well as the other walls of the church show a similar construction approach. Contrary to other church typologies, the façade does not differ significantly from the rest of the walls, showing the same construction approach. The perimetral walls appear monolithic and are characterized by a repetitive rhythm of openings, well-marked by squared stones, with characteristic crenellation at the top (Figure 5- 4). The monolithic appearance is also underlined by the interconnection between walls, well-marked recurring to squared block stones (Figure 5- 5).

(a) **(b)**

Figure 5- 4 The crenellation: **a**. View from the exterior side; **b.** View from the interior side at the level of the roof of the lateral naves (Photo rights R. Fleres).

(a) **(b)**

Figure 5- 5 Details of the masonry fabric: **a.** Interlocking between walls with squared stone blocks; **b.** wall pattern of the walls constituted by roughly squared stones with layers of bricks at a constant spacing of approximately 60 cm (Photo rights R. Fleres).

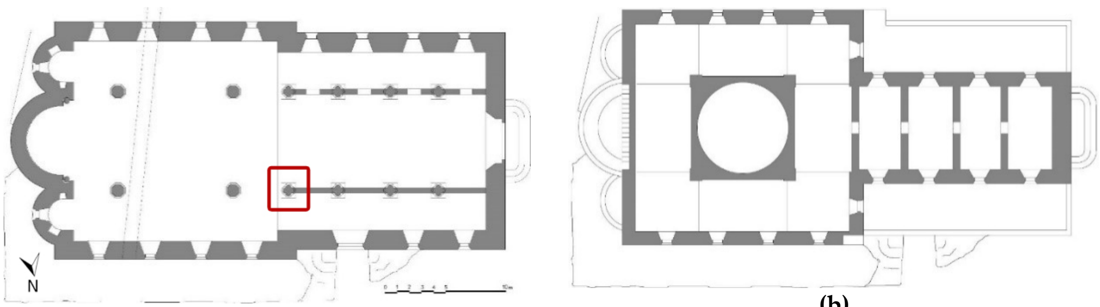
The interior of La Badiazza reflects the magnificence of the exterior. The planimetric distribution shows a division of the structure into two distinct volumes: the tripartite nave space and the transept with the function of presbytery, with three apses at its back that project from the perimetral walls.

The naves are rhythmed by eight polylobate columns per side, with decorated capitals (Figure 5- 5, Figure 5- 6). These columns bear the loads transferred by the ogive arches. In this church configuration, the system of arches plays a significant structural role both in the longitudinal and transversal direction, creating a sort of structural grid that interconnects the perimetral walls.

The presbytery is in axis with the nave space and it is one of the elements that characterizes this type of churches. The presbytery is the point of origin of the space distribution, called *qubba* in the Islamic construction culture. It was the configuration scheme for the presbytery in the churches of the Norman period. During such domination, the construction of *basilian* churches that refer to the basilian monastic order of Greek origin was allowed. In La Badiazza, the presbitery has the typical square layout well defined by four columns that support the ogival arches, which redistribute the loads of the upper structures. In the churches with this configuration, the presbytery was usually covered with a dome, where the connection between the square of the space of the presbytery and the circle of the dome, in some cases, was solved with recessed arches, a solution that derives from the Islamic culture differently from the byzantine one that recurred to pendentives (Billeci 2000). In La Badiazza, the dome is no more there, but the recessed arches above the ogival arches clearly mark the existence of this architectural element in the past. The square space described by the four pillars is confined by four pointed arches that connect this central space with the external walls in the transversal direction, and by other four that link the apse area with the central space and the central space with the naves, in the longitudinal direction. Moreover, four corner spaces that recall the women's galleries in churches, called "matronei" confine the square space. They were generally located in the nave. Being difficult to access, a possible reason of the presence of these four, one per side, could be justified through a structural function. Indeed, they seem to work as embankment of the dome, as also reported in (Basile 1975).

The plan configuration of La Badiazza, characterized by three apses in the back of a square presbytery that connects with the tripartite nave, recalls for some aspects the church of Santo Spirito in Palermo (Ciotta 1992 p.125).

The vaults of the naves are decorated with ribs with a rectangular cross-section, characteristic of the "svevian" period (Figure 5- 7). Their function could be that of covering the joints in the diagonal corners of the groin vaults, as reported in (Billeci et al. 2013.). In the main façade and in those of the apses, the openings have a round shape that recalls those of the Cathedrals of Cefalù and of Palermo. The door dates back to the 14th century. Inside the church, the capitals are naïf and are expression of the local builders, recalling the Corinthian style.



(a) (b)

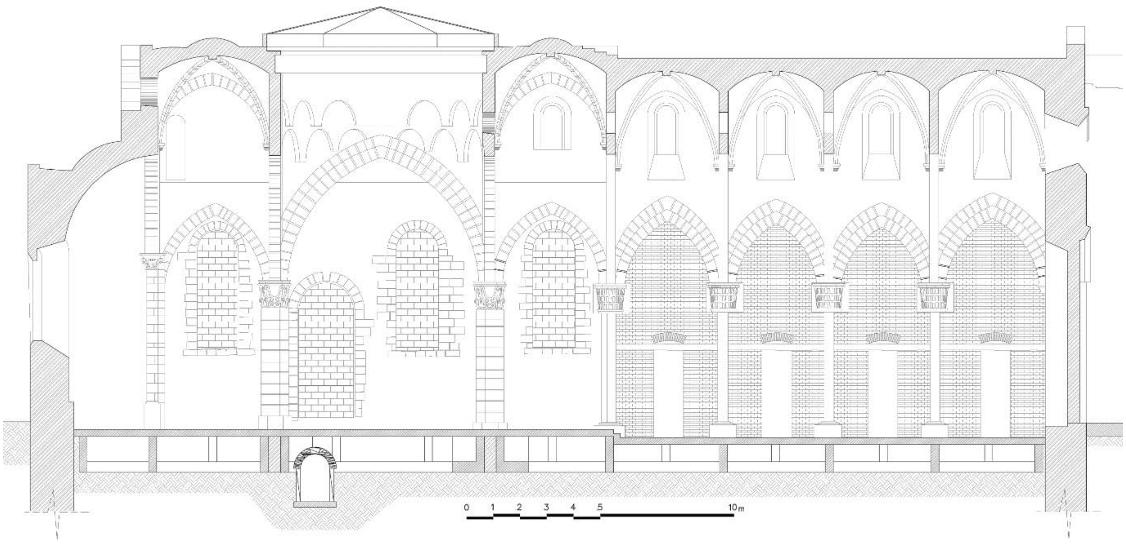
Figure 5- 6 Plans of La Badiazza church: **a.** Ground level (0.00 m). The retrofitted concrete column is highlighted in red; **b.** Clerestory level plan (+ 9.00 m).

(a)

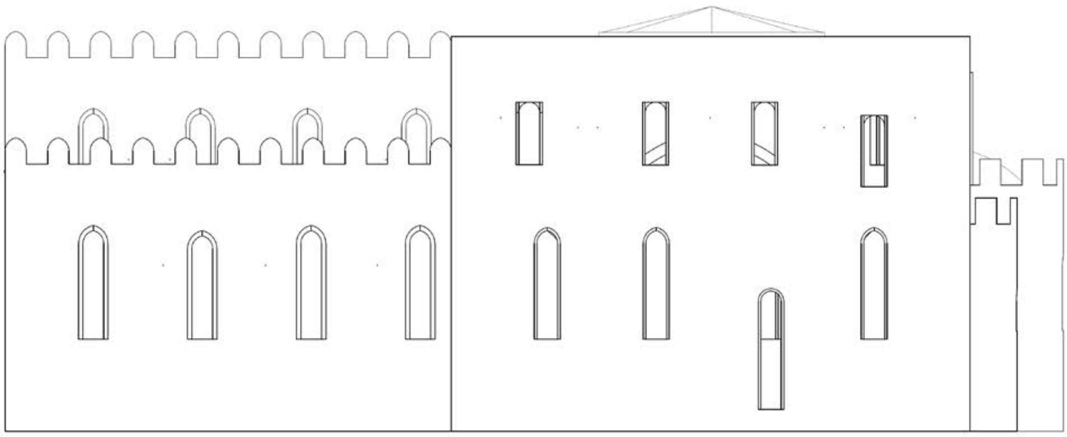
(b)

Figure 5- 7 Interior of the church: a. Pillars and arches of the nave; b. Detail of the ribs of the vaults. (Photo rights: R. Fleres)

In the same way, the windows create an extraordinary movement in the configuration of the walls of the church, visible both inside and outside (Figure 5- 8).



(a)



(b)

Figure 5- 8 The role of the openings in describing the configuration of the walls: a. Longitudinal section; **b.** South elevation.

There are several hypotheses regarding the origins of La Badiazza. In the second half of the 19th century many historians tried to date the origin of this building. Some of them arrived also to hypothesize that the church was built in two phases: a first phase that concerned the construction of the square plan of the presbytery with the three semicircular apses and the dome and a second one that developed tripartite space of the nave. Another aspect that contributes to uncertainty in the dating is in the similarities of names with other monasteries of the area (Basile 1975). The first certain chronological reference dates to 1168, corresponding to the donation by King Guglielmo. In this year, the king of Sicily, Guglielmo II, declared the church as royal chapel, giving its sovereignty to the Pope (Samperi 1644). The monastery, set on fire and partially damaged during the Vespri War, 1282, and besieged by the Angioini, was enlarged by Federico III d'Aragona, who ordered to rebuild the church. With the plague, in 1347, the monastery was gradually transformed into Summer residence. The nuns moved in a new monastery complex inside the urban walls (La Farina 1840). This episode contributed to the progressive abandonment of the building, which culminated in the 16th century. The lack of maintenance and the earthquakes occurred in the years have soon led the building to decadence. The dome collapsed between 1838 and 1840, probably due to the earthquake that affected the area in 1839. Indeed, in 1842 M. d'Azeglio painted the church but the dome does not seem to appear (Basile 1975).

Some years later, in 1851, another earthquake occurred, and the already damaged church deteriorated further. Subsequently, in 1855, La Badiazza was filled with debris because of a flood. About fifty years later, 1908, the Stretto di Messina earthquake struck the area. For the proximity of the church to the epicenter (16.79 km) and the magnitude of the event (7.10 *Mw*), this seismic event could have influenced the state of the structure, but no reliable documentation was found, which probably implies minor damage (Figure 5-9).

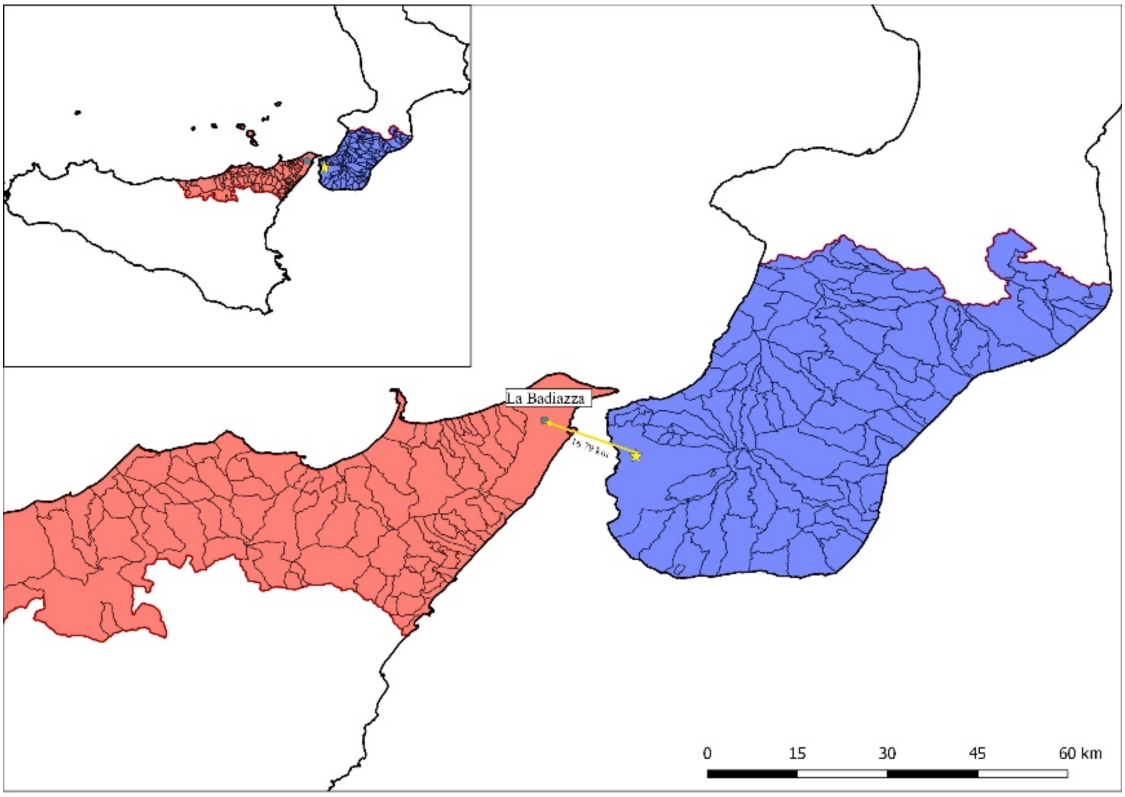


Figure 5- 9 The Stretto di Messina earthquake in1908 (7.10 *Mw*). The yellow star points out the epicenter of the 28 December 1908 event at 04:20 a.m. The blue point represents the position of La Badiazza church, 16.79 km far from the epicenter. The blue region is Reggio Calabria and the pink one is Messina.

In the first half of the 16th century, La Badiazza underwent some strengthening works: the intercolumns of the north side were infilled with roughly squared stones and hydraulic mortar. Only in more recent times, the south side was rebuilt with solid bricks and cement mortar. Between 1951 and 1955, the Superintendence of cultural heritage allowed to build a concrete wall at the site of the church, to protect the historic building from other possible floods.

Extensive information is available on the studies and the interventions carried out on the church in 1984. A sonic test was performed on the walls, confirming the homogeneity of the materials, which led to exclude the presence of structural modifications. The endoscopic tests carried out on the masonry, up to 40 cm in depth, confirmed a uniformity in the way it was built: the masonry is made of mixed stones with pebbles of different diameters, with mortar joints of homogeneous characteristics and layers of bricks. On their side, the ultrasonic tests recorded lower speeds compared to the speed that may be usually associated to the masonry type under examination. This aspect justifies assuming the presence of small inhomogeneities, such as a greater porosity or micro-cracks. Further studies confirmed an inadequate static behavior of the columns and arches of the nave. Moreover, in the same period, the first intervention of strengthening of the foundation system was carried out. Because of these tests, the columns were disassembled, each block of stone reworked on the contact surfaces and those in poor static conditions replaced with others made of Comiso stone, having mechanical characteristics close to the original ones. In order to respond to the high stress demand at the base of the columns, tubular profiles were locally inserted. Finally, the arches above the columns were consolidated with injections of mixed mortar to support the weight transmitted by the infill walls between the arches, while the ferrules were consolidated with aramid fiber connectors of 8 mm, radially set. Figure 5- 10 summarizes the series of historic events, constituting a reference in the process of understanding the interventions executed on La Badiazza.

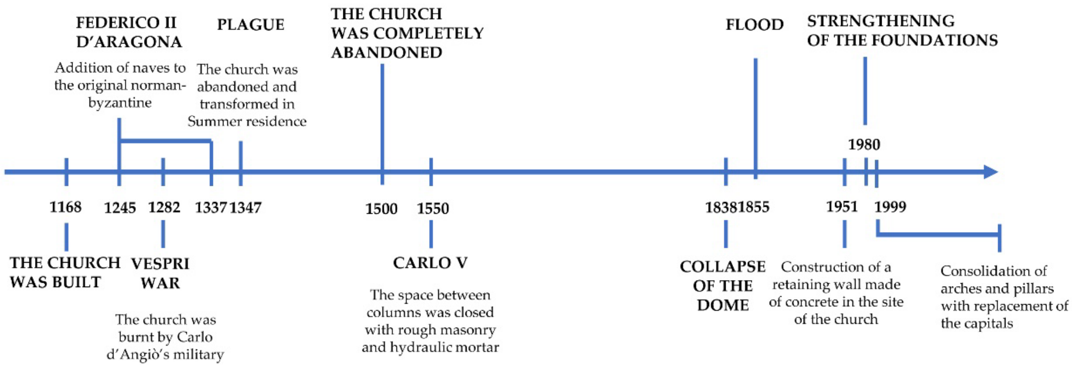


Figure 5- 10 Timeline of the principal events which have affected the history of the church.

Tie rods can be found in different parts of the church (Figure 5- 11). Iconographic documents show the tie rods already in place before the collapse of the dome and before the flood occurred, which was in the first half of the 19th century. The effectiveness of such tie rods has not been checked by diagnostic tests, but it has been tentatively assumed in this work.

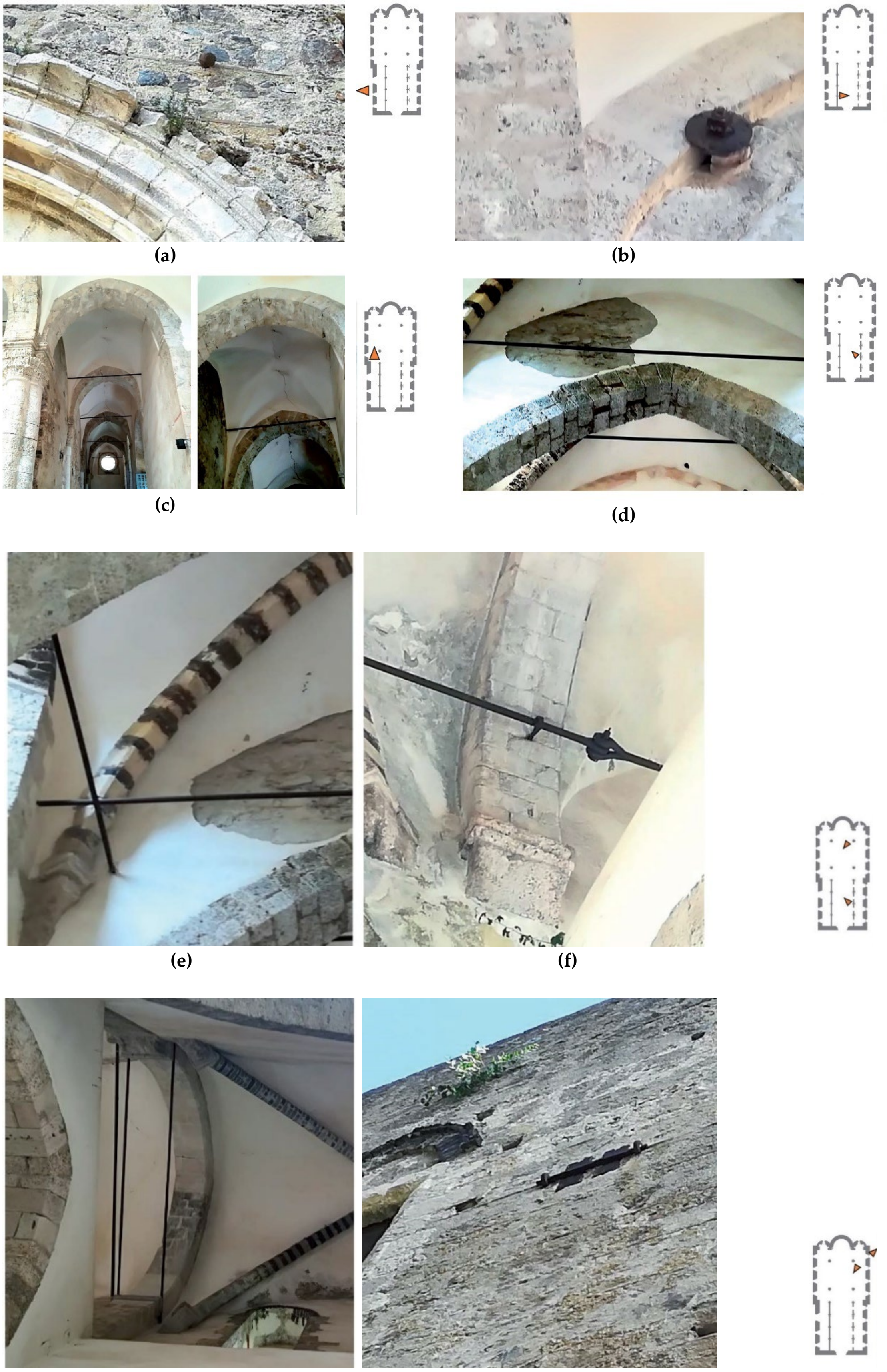


Figure 5- 11 Tie rods and wall anchors (“capichiave”): **a.** Detail of the wall anchor of the longitudinal tie rods; **b.** Detail of the wall anchor from the interior; c. View of the lateral nave with tie rods; **d.** Two tie rods in the lateral naves that pass close to the arches of connection between the vaults of the lateral

naves; **e.** Detail of transversal and longitudinal tie rods; **f.** Detail of connection of the tie rods; **g.** Tie rods in the arches of the transept; **h.** Wall anchors of the arches (Fleres 2017).

Figure 5- 12 shows the overlapping of the seismic events of the area with the principal events for the church building.

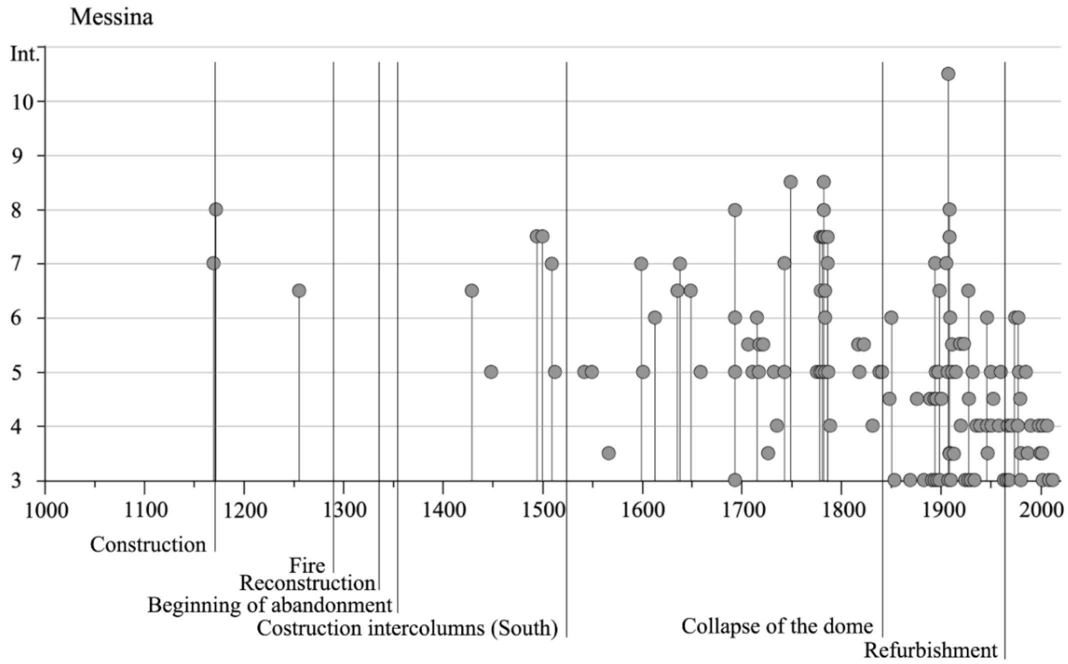


Figure 5- 12 The histogram represents in the abscissas the year of occurrence of an earthquake and the principal events of the church and on the ordinate axis the intensity value. The most significant events in the building life have also been reported (Fleres 2017).

5.2.1 The tri-dimensional architectural model

Before implementing the numerical model (Section 5.2.2), a three-dimensional architectural model was created, in order to get a deeper insight of the complexity of the structure. The model shows the configuration that the church has today, that is, with the intercolumn space closed by a brick curtain wall and without the masonry dome. Figure 5- 13 explains, recurring to different colors, the main constitutive elements and the distinct materials present in the church. The external walls are made of squared calcareous local stone units with gypsum mortar joints, similarly to the polylobate columns. An exception is the last column of the right lateral nave, which was replaced with a concrete core one. The pointed groin vaults are covered with an infill material made of irregular stones with lime and river sand. Such three-dimensional model is a simplification, needed for the creation of the numerical model. An example of such approach is represented by the columns, simplified if compared with the real polylobate ones.

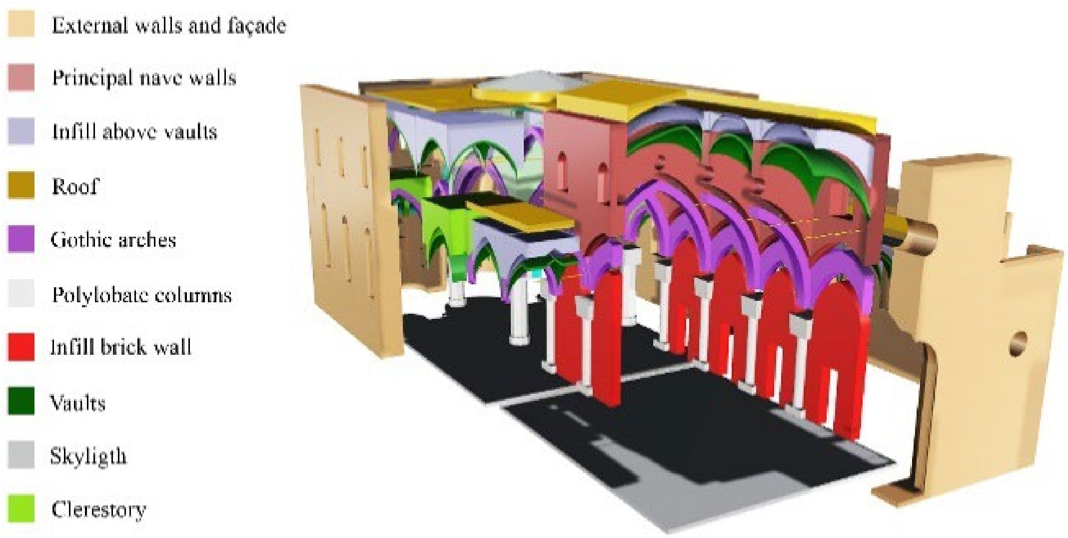


Figure 5- 13 Three-dimensional model of La Badiazza (Fleres 2017).

5.2.2 NUMERICAL RESULTS

In this section, the main numerical results are presented in order to understand the structural behavior and to trace some considerations on the seismic vulnerability of churches of the same typology.

Two numerical models were developed in order to appreciate the effects of the building modifications on its structural behavior: one reproducing the existing configuration, without the dome and with the infilled intercolumns (Model 1), the other representing the 16th century configuration without the infill in the colonnade and with a hypothesis of dome (Model 2). Model 1 and Model 2 are made of plate elements for walls and vaults and beam elements for the columns. For the analyses an equivalent homogeneous isotropic material was chosen with a Young's modulus equal to 1620 KN/m^2 and a mass density of 3600 Kg/m^3 for the masonry walls, referring to the values reported in the Commentary of the Italian Building code (CMTI 2019). For the concrete column, an elastic modulus of 31000 MPa, a compression strength (σ_{cu}) of 14.2 MPa, and a tensile strength equal to 1.2 MPa, referring to the values reported in the NTC 2018 for a C25/30 were used. Other detailed information on the detailed procedure followed to obtain the numerical results is consultable in Fleres (2017).

Both configurations show the global behavior of the structure with a participating mass that reaches the 80% in the transversal direction and the 68 % in the longitudinal one already at the 15th mode (Table 5- 1). In general, the structure responds to actions as a whole; only in the upper portion of the walls, particularly at the level of the crenellation, local behaviors are visible (Figure 5- 14).

Detailed consideration of the modal results leads to the following observations. Mode 1 shows a general displacement trend in the transversal direction (y), with displacements having the same sign. The maximum deformation is in the connection area between the transept and the naves. The participating mass reaches 60.76%, a high value more typical of other building typologies, which indicates that in this direction the building presents an important global behavior. The mass involved is mainly in the central area: in this first mode, the façade and the apse are not engaged or only mildly engaged in the displacement.

Mode 2 is mainly a torsional mode around the vertical axis, z, involving 36.92% of the mass. The deformed shape presents an inflection point at the connection of the naves with the transept. The maximum deformation is in the transept, similarly with what will occur in Mode 10.

In Mode 3, the displacement is, again, mostly concentrated along the transversal direction (y), summing up to a total participating mass ratio of 74%. The maximum deformation concentrates in the lateral naves at the level of the connection between the vaults and the lateral walls, suggesting the possible development of an out-of-plane failure mechanism of the lateral walls in this portion of the church building.

The following series of modes do not involve the longitudinal direction, which will resume importance from the 10^{th} mode onward, building up the total participating mass with moderate contributions. Mass ratio will reach a remarkable value of 80% with the 15^{th} mode.

Modes 4 and 5 are in the longitudinal direction and reach a total participating mass of 62.3 % in that direction, which will slowly increase to 67.6 at the 15th mode, with none of the modes prevailing.

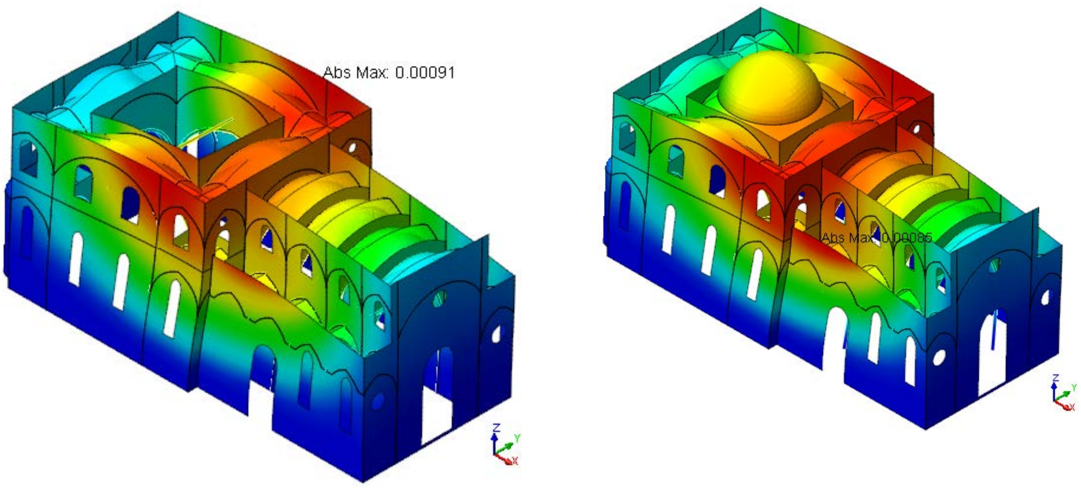
In Mode 4, the deformation involves only the transept. In particular, the maximum deformation is concentrated in proximity of the lateral vaults of the transept. The transept walls move out-of-plane, inwards for the end wall and outwards for the lateral ones, or vice-versa. At the top of all these walls an overturning mechanism may be expected with the lower border connecting the wall openings or starting from them. The maximum modal displacement occurs at the top of the end wall.

Mode 5 engages all the structure and, indeed, presents a participating mass ratio of over 40%. It implies an important deformation of the vaults of the transept and of the corresponding lateral walls. Deformation patterns for these elements once more suggest an evolution into limit mechanisms.

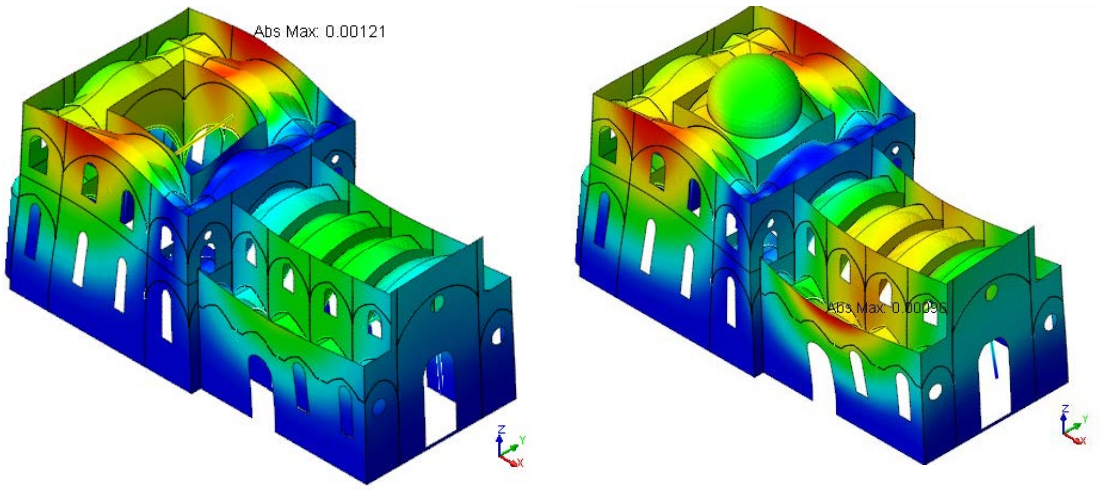
Mode 6 is mainly torsional and shows a deformation in both principal directions of the church, involving in particular the two vaults of the transept, with the corresponding lateral walls bending in parallel. Mode 7 involves only the upper part of the lateral walls of the nave showing a possible overturning mechanism of the crenellation. In mode 8 the main façade and the end wall of the apse move in the longitudinal direction with opposite sign, similarly the lateral walls of the transept move in opposition. The maximum displacement appears in the vault against the wall of the apse. Mode 9 shows a rotation around z and the maximum deformation concentrated on the vaults of the apse.

Table 5- 1 Periods and participation mass ratio from the modal analyses performed in Model 1 and Model 2.

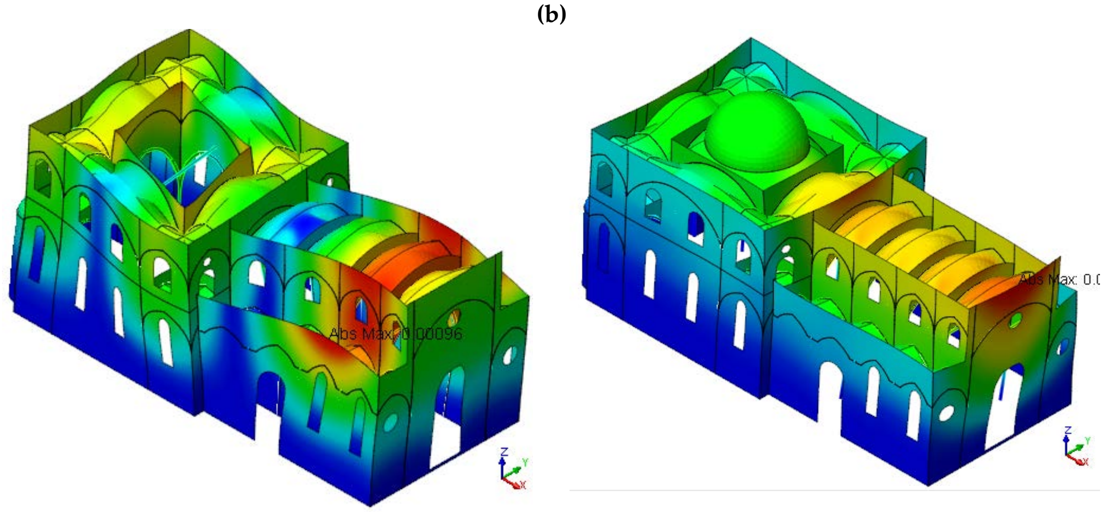
	Model 1			Model 2		
Mode	**(%) Participating mass ratio (Longitudinal)**	**(%) Participating mass ratio (Transversal)**	**(s) Period**	**(%) Participating mass ratio (Longitudinal)**	**(%) Participating mass ratio (Transversal)**	**(s) Period**
1	0.00	60.76	0.542	0.00	64.95	0.554
2	0.00	1.05	0.357	0.27	0.20	0.349
3	0.09	12.22	0.290	62.70	0.00	0.339
4	22.18	0.01	0.282	0.00	10.85	0.274
5	40.05	0.02	0.269	4.07	0.02	0.267
6	0.00	0.07	0.218	0.03	0.00	0.230
7	0.25	0.00	0.218	0.70	0.00	0.210
8	0.46	0.00	0.209	0.30	0.00	0.209
9	0.11	0.40	0.196	0.00	0.21	0.203
10	0.07	4.31	0.175	0.75	0.00	0.177
11	2.39	0.21	0.174	0.00	0.83	0.169
12	1.37	0.00	0.162	0.00	1.43	0.167
13	0.56	0.01	0.159	0.00	2.50	0.159
14	0.02	0.18	0.154	0.35	0.02	0.155
15	0.08	0.91	0.152	0.03	0.00	0.149



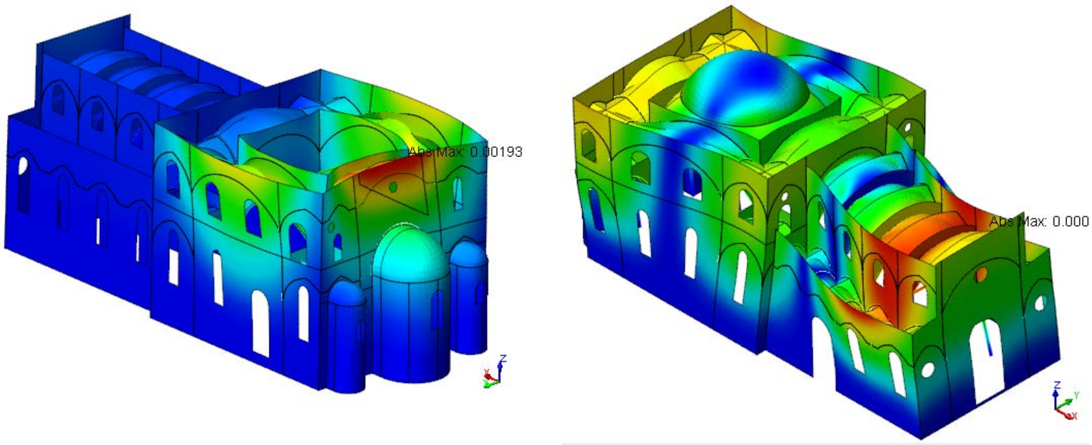
Mode 1
(a)



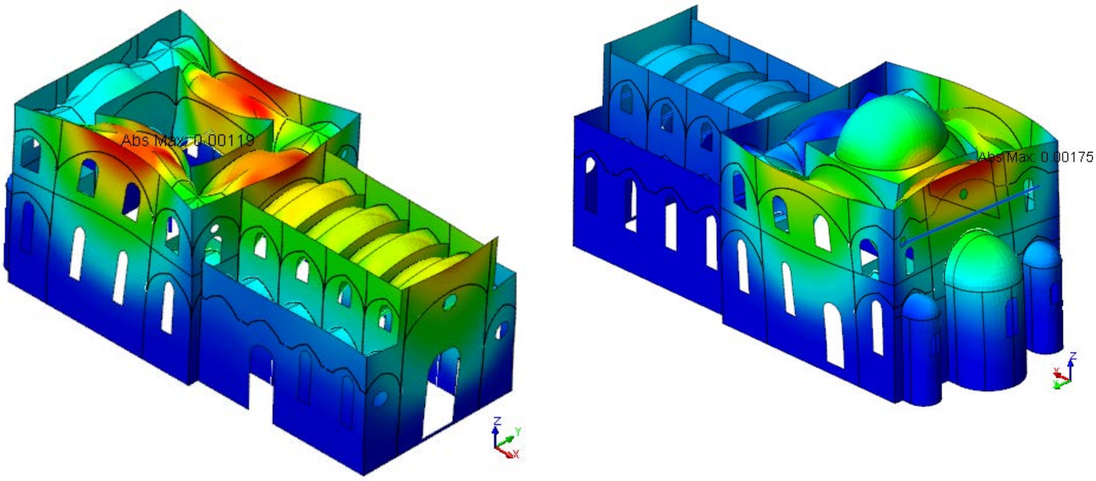
Mode 2
(b)



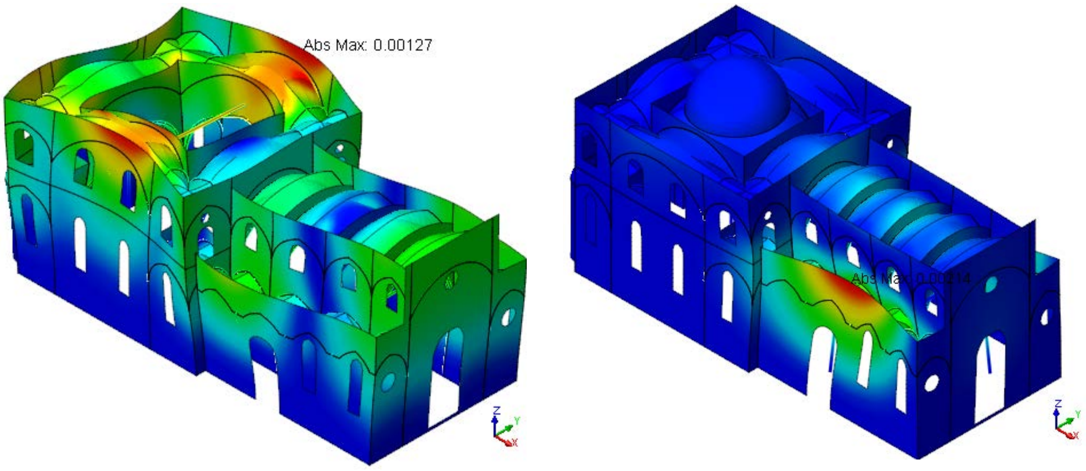
Mode 3
(c)



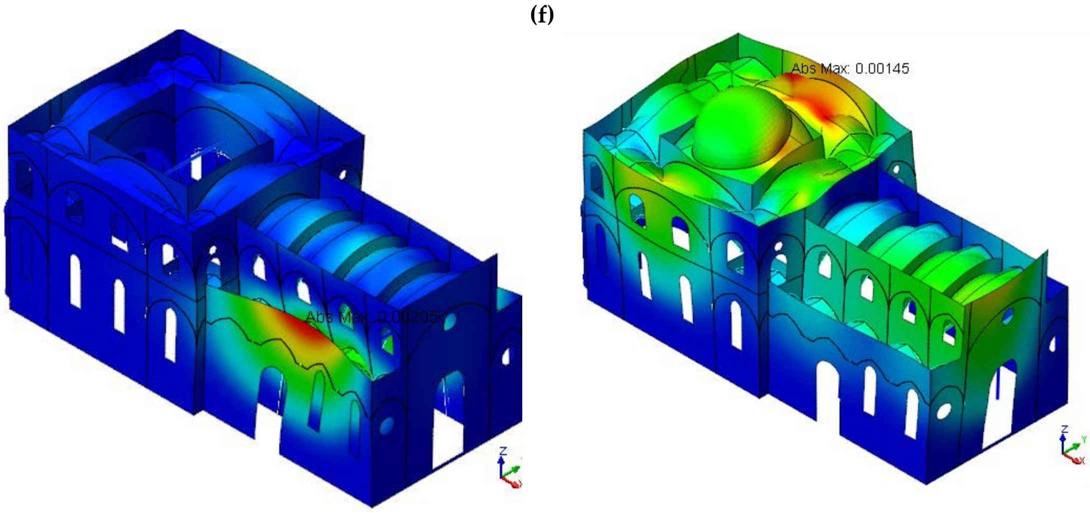
Mode 4
(d)



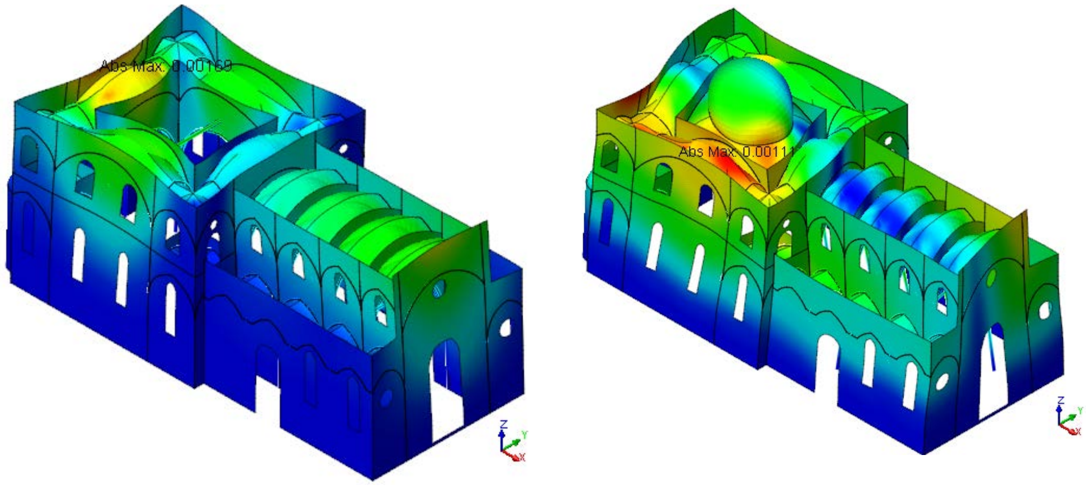
Mode 5
(e)



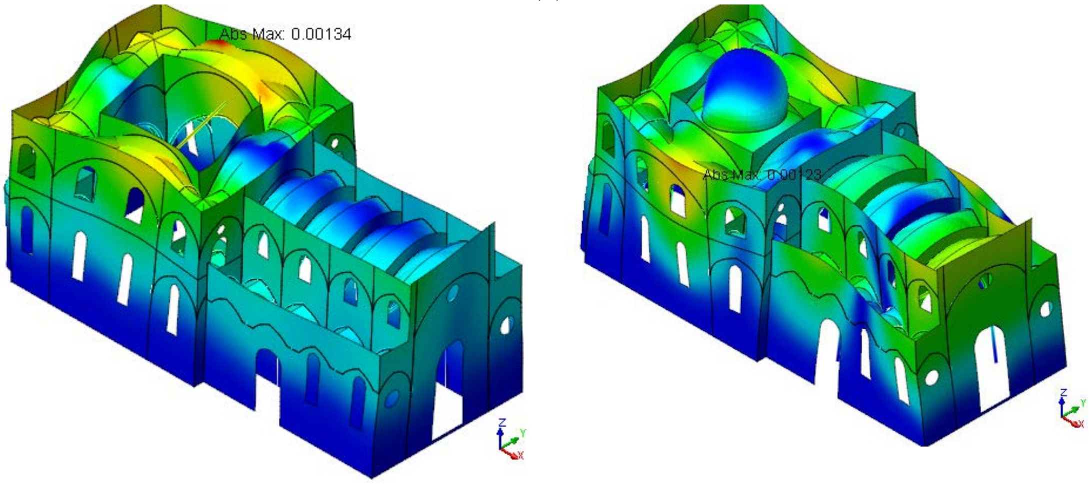
Mode 6
(f)



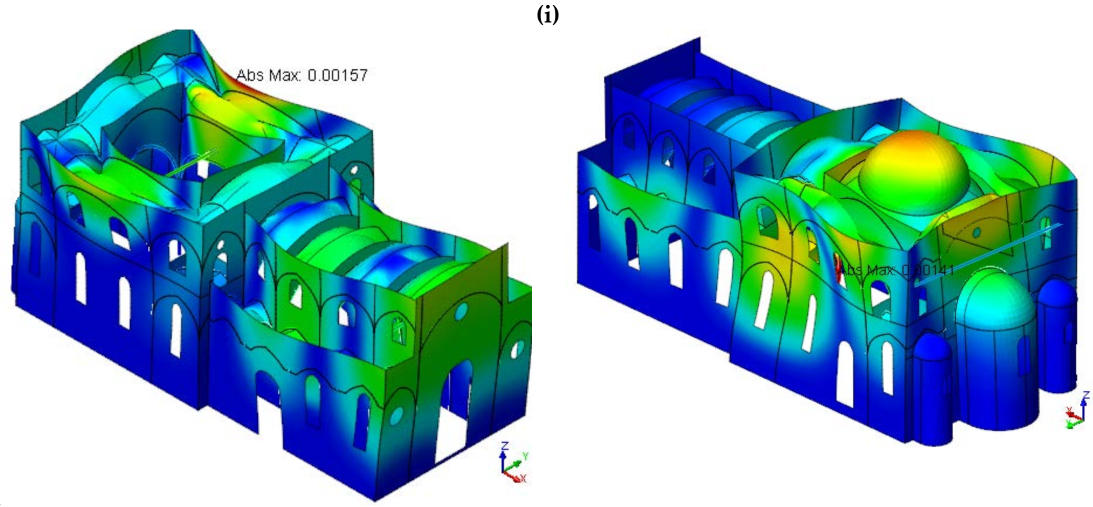
Mode 7
(g)



Mode 8
(h)



Mode 9
(i)

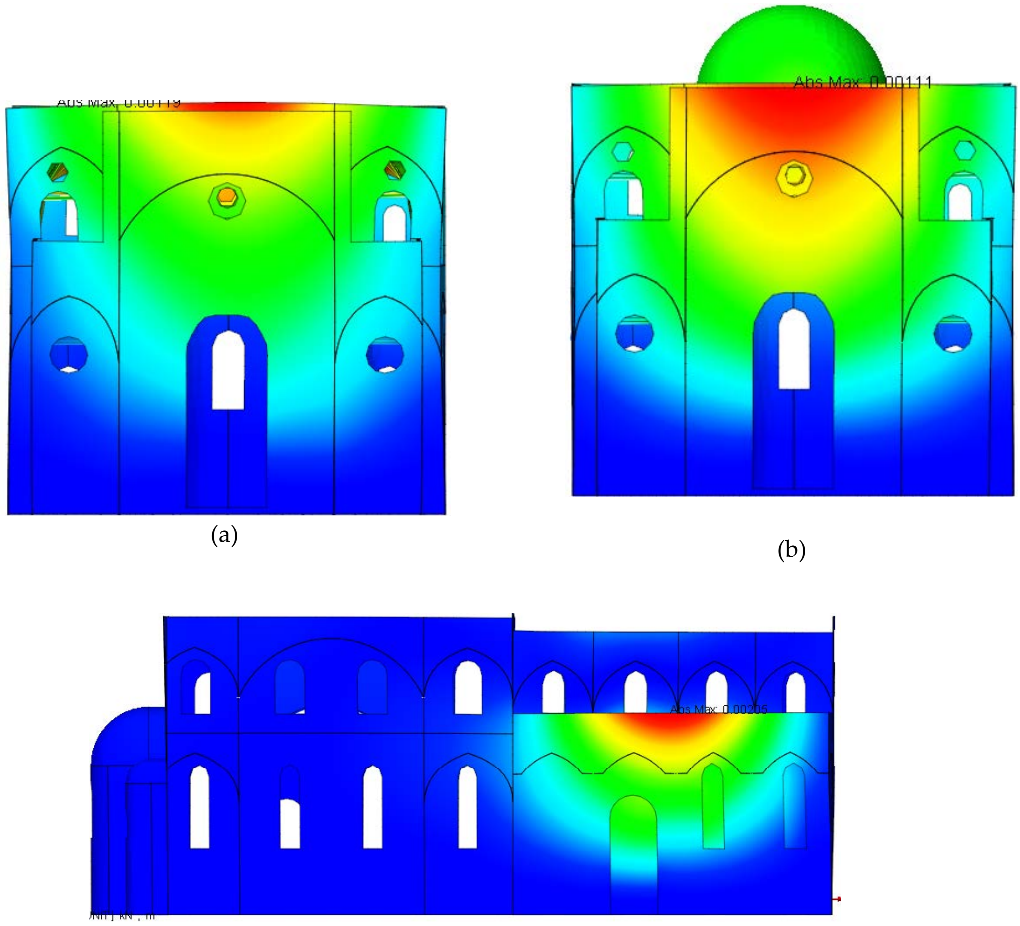


Mode 10
(l)

Figure 5- 14 Modal shapes of the two church configurations (on the left Model 1 and on the right Model 2): **a.** Mode 1; **b.** Mode 2; **c.** Mode 3; **d.** Mode 4; **e.** Mode 5; **f.** Mode 6; **g.** Mode 7; **h.** Mode 8; **i.** Mode 9; **l.** Mode 10.

The comparison between the two structural configurations clearly shows the similarity of the first two modes. The third, however, differs: the second structure presents here an almost purely longitudinal mode, with high participating mass ratio, about 63%, which rapidly balances participation in the two main directions. Due to the addition of such mode, subsequent modes of the second structure grossly correspond to those of the first, shifted by one.

In both cases, two characteristic mechanisms for this typology can be identified: the façade mechanism limited to the upper part, with the typical V shape (Figure 5- 15a, Figure 5- 15b), due to the good connection of the façade to the longitudinal walls that is assumed in the model and reasonably expected in reality, as well as to the presence of the rose window, and the overturning of the crenellation of the lateral naves (Figure 5- 15c).



(a)

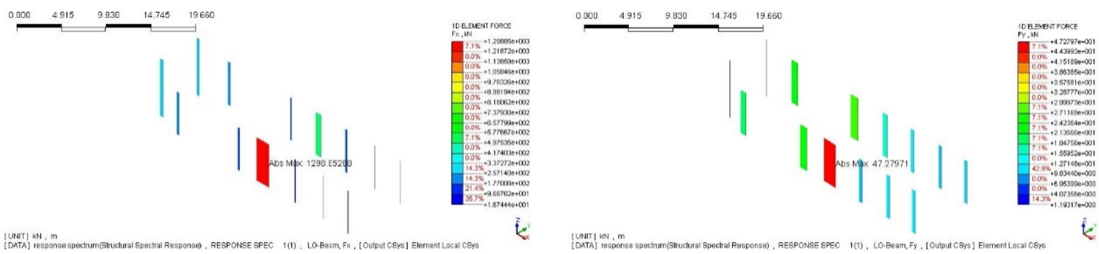
(b)

(c)

Figure 5- 15 Out of plane mode: **a.** façade in Model 1; **b.** façade in Model 2 (Mode4); **c.** crenellation lateral walls of the naves in Model 1 (Mode 7).

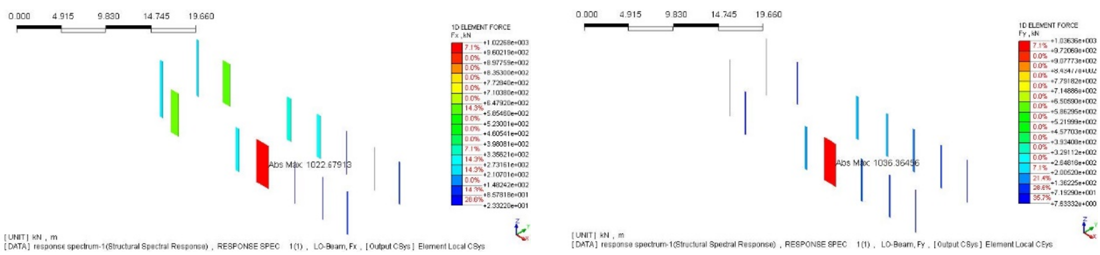
A kinematic limit analysis of such local mechanisms has been performed; such analysis is fully developed in Fleres (2017).

Response spectrum analyses of the church in the current configuration were performed, similar to other works in the literature (e.g. Brandonisio et al. 2013) with the seismic action applied in the longitudinal and transversal directions, in order to examine the stress distribution on the various elements and, in particular, to investigate the implication of the column realized with a concrete core. The distribution of internal forces in the column system appeared well balanced, except for the stiffer concrete column, which naturally concentrates a much higher stress level. This effect may be appreciated comparatively in the images reported here, which summarize the results obtained by applying the earthquake in the longitudinal direction (Figure 5- 16) and in the transversal one (Figure 5- 17).



(a) (b)

Figure 5- 16 Earthquake in x direction (longitudinal): **a.** shear Fx; **b.** shear Fy.



(a) (b)

Figure 5- 17 Earthquake in y direction (transversal): **a.** shear Fx; **b**. shear Fy.

5.3 CONSIDERATIONS

Some final conclusions for this chapter can be drawn, part of which strictly related to the case study, others applicable to churches with similar configuration.

La Badiazza presents some elements that date back to particular periods of its life history and that influence its vulnerability. The analyses have quantified how the column with the concrete core concentrates the stresses and have shown how the infill of the intercolumn space provides greater stiffness to the structure in the longitudinal direction. The tie-rods, regularly applied at each span on the arches and vault, unify their behavior producing a collaborating arch-vault system that supplies stiffness to the church in the transversal direction. The presence of a dome would probably improve to some amount the longitudinal response, but this requires assuming a fully effective interaction between the dome and the rest of the structure of the church. The loss of such structure, occurred in the past, indicates in reality possible criticalities related to construction issues that should be investigated in detail on other similar domed structures. The mesh of arches in the transept seems capable of producing an effective three-dimensional frame system.

In a wider perspective, some considerations can be expressed in relation to this church configuration that appears similar to a fortress and has a planimetric layout that may be found in other churches of the XI and XII in Sicily. Globally, this church structure is characterized by a significantly global behavior, similarly to the case of a spine wall building. In such church typology the weakest walls are the lateral walls of the transept that are the only ones with a relevant height, therefore very deformable. The church structural behavior is symmetrical along the longitudinal axis. In such configuration, the façade, which is often a highly vulnerable element in other church typologies, has here the same order of importance as the rest of the walls; the presence of the rose window in the middle of the façade, together with a good connection to the longitudinal walls, indicates that the "V" shape overturning mechanism is more likely than a more devastating total wall overturning. Internally, the structure is constituted by a series of arches that create a sort of frame system that interconnects the perimetral walls and distributes the upper loads. In the presbytery the arches play also a significant role in supporting the loads of the recessed arches above and coming, in turn, from the dome. Such specificity seems to reduce the vulnerability of the structure because the behavior tends to be enclosed in specific zones of the structure. Finally, the presence of crenellations, which in the modal shapes have shown to be susceptible to overturning modes, adds an element of vulnerability to this typology. Another interesting characteristic in these churches is the presence of three semi-circular apses that provide significant stiffening in the longitudinal response, acting like buttresses. However, these assumptions need to be proven with further non-linear analyses.

CHAPTER 6

THE PROPOSED APPROACH

This section is devoted to explaining in more detail the proposed approach, looking at the territorial specificities.

The critical issues pointed out for vulnerability assessment in the first part of this research (Introduction and Chapter 1) and the observations on specific aspects from the sample churches from various Italian regions examined in Chapter 3, 4, 5 have suggested to investigate the possibility of a different approach. This approach should acknowledge the specificities, typical of a territory, that have influenced the construction and the subsequent modifications of the church building.

6.1 OBSERVATIONS AND CONCLUSIONS FROM THE STATE-OF-THE-ART REVIEW

The main inconveniences that may be met in a vulnerability survey are usually related to the impossibility to reasonably match the case under exam with the reference cases given in the classification procedure. Referring to the original Italian approach, these difficulties may be summarized in the following points:

- *Low flexibility*. The list of the possible kinematic mechanisms used as reference is currently limited to 28, associated with a series of pre-identified macro-elements. The selection of these macro-elements and mechanisms derives from the analyses of an extremely broad collection of damage observations and represents well most of the cases that may occur. Nevertheless, in the presence of macro-elements that do not correspond to the pre-defined ones, the surveyor does not have the possibility to report the information. Consequently, such item is excluded from the examination and, specifically, from the calculation of the index of vulnerability.

- *Low adaptability of the procedure to a different context*. The procedure is difficult to be used outside the Italian context, where it was formulated. Such issue was observed, for instance, by various authors in Chile, in the Maule earthquake, 2010, and in the Philippines, in the Bohol and the Typhoon Haiyan earthquakes, 2013, (D'Ayala and Benzoni 2012; D’Ayala et al. 2016), as well as in New Zealand, in the Canterbury earthquake, 2010-2011, (Leite et al. 2013; Lagomarsino et al. 2019), and in the Azores earthquake, 1998 (Magalhães et al. 2012). In Chapter 7, the same limitation was met during the application of the original methodology to the seismic vulnerability assessment of some churches in Montreal Island, in Canada.

- *Damage-oriented procedure.* The seismic vulnerability assessment procedure was created from damage observations and the same procedural layout of damage assessment operations was maintained. The surveyor goes through a list of 28 limit mechanisms to assess the church and to provide weighted coefficients, which, when summed up, give an index of vulnerability (MiBAC 2011). At territorial scale, such an index provides indications for statistical purposes and for risk assessment, but it tends to flatten important indications on the vulnerability for the group of surveyed churches. Some useful information on vulnerability derived on local construction practice may be lost. Nowadays, the long seismic history in Italy has induced the development of specific studies that have deeply investigated the factors generating vulnerability and a change of perspective may be attempted. A proposal is detailed in Section 6.3.

6.2 OBSERVATIONS AND OUTCOMES FROM THE ANALYSED STUDY CASES

The possibility to work on two different Italian areas, that is, Marche, Eastern Lombardy and to treat a specific case of church-fortress in Eastern Sicily, has facilitated the understanding of the influence of local construction culture on the church configurations and structural behaviour. In the first two cases, the study of earthquake damage has allowed to understand the vulnerability that characterizes the building at the time of the event. In the last case, the implications of interventions and historical modifications on the structural response of a historical church specific in this church configuration have been analysed. The church, representing a well characterized period of the Sicilian history and built according to the corresponding architectural style, underwent restoration according to an approach typical of the 1980s. Both these aspects define this building and determine its seismic vulnerability.

The previous chapters have suggested a change of perspective, inducing to face the issues of the seismic vulnerability of churches through a territorial approach. Aspects that can change from one area to another and affect the vulnerability may be:

- *The different seismicity.* The Italian territory is characterized by a very diversified seismicity, in terms of frequency and intensity of the seismic events. These aspects influence the approach towards the damage prevention strategies. As specified in the introduction, studying different geographical areas has led to distinguish the historical awareness from the modern one. Indeed, these aspects have shown to influence the structural configuration of the building. An example is the eastern Lombardy with only a limited presence of anti-seismic details and provisions, like tie rods and buttresses in historical construction, because of a low awareness at the construction practice level, as the earthquake of 2012 has evidenced.

- *Different history of territories* that has supported the development of certain architectural styles characterized by a specific configuration and has spread the associated vulnerability.

- *Different availability of materials* that has determined the development of techniques of construction and of skillful craftsmanship in that territory.

- *Different approaches to interventions according to design Codes:* in modern times, repair and strengthening interventions are guided by design codes, which are upgraded in time. Codes have been applied for repair at different times in various areas, depending on the time of earthquake occurrence. The date and modality of interventions affect the current vulnerability.

Studying the seismic vulnerability of churches from a territorial perspective allows to collect information on the local specificities. If this information is collected and properly organized, it contributes to increase the knowledge on the elements of vulnerability of the territory concerned. Indeed, considering the church sample with the existing original

methodology the territorial specificity knowledge is blurred and information that, if recognized, may result useful for damage prevention strategies is neglected.
Some specificities, resulting from this study are reported here:

- For *Marche:*
    - Presence of thin brick vaults: they have been added later for an architectural style trend of the 18th century. This adds an element of vulnerability to the church building;
    - Belfries were usually rebuilt after severe damage or collapse, because of the seismicity of the area and the vulnerability of such architectural element;
    - Large presence of tie rods mainly in the transversal direction;
    - Bell tower frequently adjunct to the triumphal arch;
    - Presence of high impact interventions executed before the Umbria Marche earthquake, 1997 (e.g. construction of concrete roofs, concrete walls);
    - Substitution of heavy roofs that had been reinforced as above, with lighter solutions, after undergoing severe damage in the 1997 earthquake.

- For *Eastern Lombardy:*
    - Large use of thin brick vaults;
    - Reduced vulnerability of the thin masonry vaults interrupted by arches or ribs;
    - Absence of tie rods, especially in the longitudinal direction, resulting in easy overturning of the façade;
    - Presence of vault adjacent to the façade, that is, a vault in direct contact with the façade or indirectly in contact through a stiffening arch;
    - Soaring gables and pinnacles.

- For the church-fortress in *Eastern Sicily*:
    - Arabic Norman churches tend to present an important global behavior;

- The vulnerability of this church typology appears relatively low;
- Vaults, domes and colonnade are the most vulnerable macro-elements for this church typology;
- Local stiffening interventions once more show to be a source of vulnerability: the insertion of a concrete column concentrates stresses and increases the vulnerability.

All these aspects clarify the strict correlation between the building and the territory, showing the importance of studying the problem from a territorial perspective, intended as defined in this research. A specific form (see section 6.4) for collecting the territorial specificities has been developed, to become a facilitator for the application of such territorial approach.

6.3 THE PROPOSED TERRITORIAL APPROACH FOR THE SEISMIC EVALUATION OF CHURCHES

In a phase of vulnerability assessment, the available time and the possibilities to set strategies of damage prevention are determinant. The approach proposed here is based on the concept of *prevention* intended as a series of integrative actions that aim at achieving a background of knowledge for the churches, as historical buildings of the same territory. For such a reason, the approach proposed is called *territorial knowledge approach*. Distinguishing features of territories, such as materials and techniques of construction, common church typologies or styles associated to local history, and the seismic awareness were considered for elaborating the proposed methodology. This is intended for a medium scale action, which means a procedure addressed to those Authorities which operate in the territory and that have a mediator role between the practitioners, who work in it, and the national codes and guidelines for preventive actions. With such an approach, the process of knowledge acquisition is directly connected with the process of interventions and damage prevention. Thus, the proposal consists of an approach rooted in the territory and its characteristics. Working during a "peace time" without the strong constraint conditions of emergency, has the advantage of time availability, which makes this approach possible. The availability of time to set strategic plans, strictly oriented to the knowledge of the seismic vulnerability of churches for the specific territory, plays a fundamental role. Such considerations, together with the awareness of the territorial influence on the structural response of churches have boosted a change of approach, a sort of inside-out action (Figure 6- 1). Already in the 1990's, in different part of Italy, some codes of practice were written, stressing the attention on the diffused construction practice proper to each territory (Giovanetti 1992, Mannoni 1995; Giovanetti 1997; Giuffré and Carocci 1997; Giuffré and Carocci 1999; Guerrieri 1999). This indicates awareness of the need to gather the construction knowledge developed by the territory.

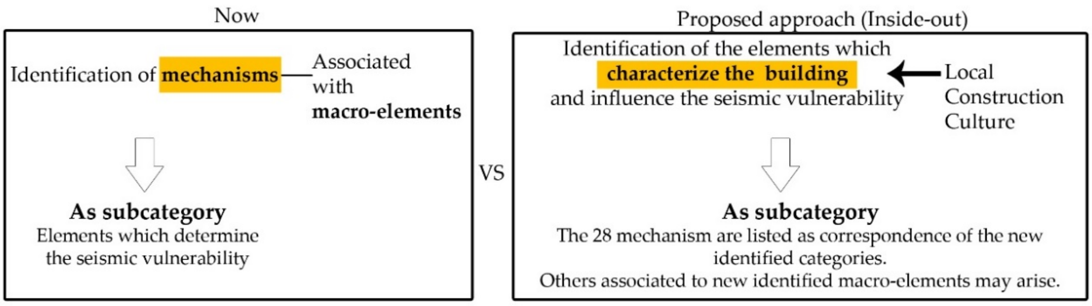


Figure 6- 1 Comparison between the actual approach and the proposed one.

In this research, an attempt to include the territorial specificities in the actual approach for each mechanism was first performed. Yet, the procedure resulted redundant in re-inserting the specificities for each mechanism, time-consuming for the surveyor, and not effective in the perspective of pointing out what makes churches of a same territory similarly vulnerable to seismic excitation.

The original approach starts from the identification of the mechanisms possible for the church under exam from a list of 28; for each of them, elements of vulnerability or the presence of seismic protection details are pointed out.

The territorial knowledge approach, on the contrary, starts first with the identification of the specificities that characterize the building and that have demonstrated to influence the vulnerability of churches. In a second step, these characteristics are associated with mechanisms proposed in the original method, as exemplified in section 6.4. This change of perspective proposes a solution to the low flexibility, adaptability to different contexts, and to the need for a vision strictly oriented towards the knowledge of the building in view of its seismic behaviour.

The application of such methodology is characterized by:

- considering knowledge of the territorial specificities as starting point;
- being aware of vulnerable elements in churches of a same territory;
- studying specific strategies of intervention and prevention at the medium scale;
- generating a flexible, updatable and more inclusive form that includes the diversity characterizing churches in different territories.

The survey form, here named *Territorial Specificity Knowledge Form* (TSK-Form), is not intended to be exhaustive, but it was conceived to be an exemplification of the approach. Indeed, teams of surveyors, organized and managed by local Authorities and their offices, such as the Superintendence or the Diocese, applying the proposed approach to the territory to which they are assigned, can customize the survey form according to specific needs. The result will be a detailed TSK-Form perfectly adapted to the territorial case, that will have the same layout for all the churches of that territory. The information collected in the form will be the basis for defining intervention strategies and design indications for local practitioners. Figure 6- 2 is a synthesis of the positive impact of the approach.

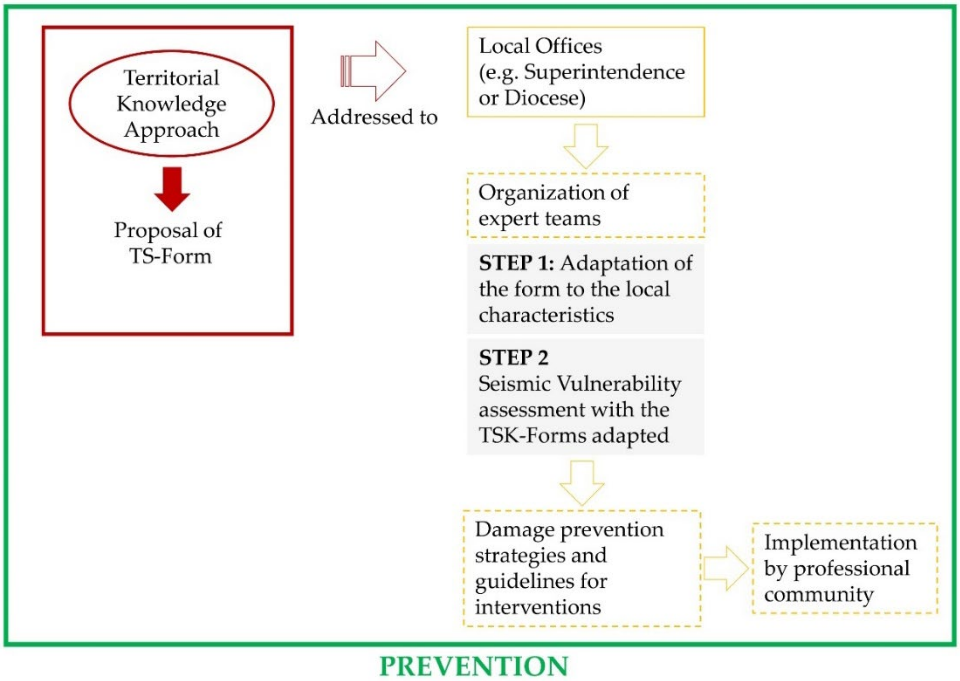


Figure 6- 2 Synthesis of the actions that the development of the territorial knowledge approach could generate.

6.4 THE INTRODUCTION OF THE NEW TSK-FORM

The TSK-Form is a proposal for a template to be used for guiding the vulnerability assessment. As a future development, it is planned to elaborate a manual that could help the assessment procedure when recurring to this form. It is an example of the application of the philosophy at the base of the territorial knowledge approach. It is intended to have a flexible format, open to future changes and adaptations. It allows, on the one hand, to assess the seismic vulnerability of the analyzed church, and on the other hand, to collect information on the significant territorial specificities that have shown to influence the seismic vulnerability of churches. The church is not only assessed but "interpreted" in the territory and in its relationship with it. The church is first seen as an object that is the result of the local construction culture and, then, as an ensemble of macro-elements. This process is the main characteristic of the proposed approach and it provides a wider comprehension of the church in relation with the territory and the local construction culture.

The TSK-Form is composed of:

- <u>A general section</u> (Figure 6- 3). This section is pre-compiled by the local office, which manages the survey. This section collects information to contextualize the church in the territory. At the bottom of this part of the form, a legend anticipates the reading layout of the detailed section of the form;

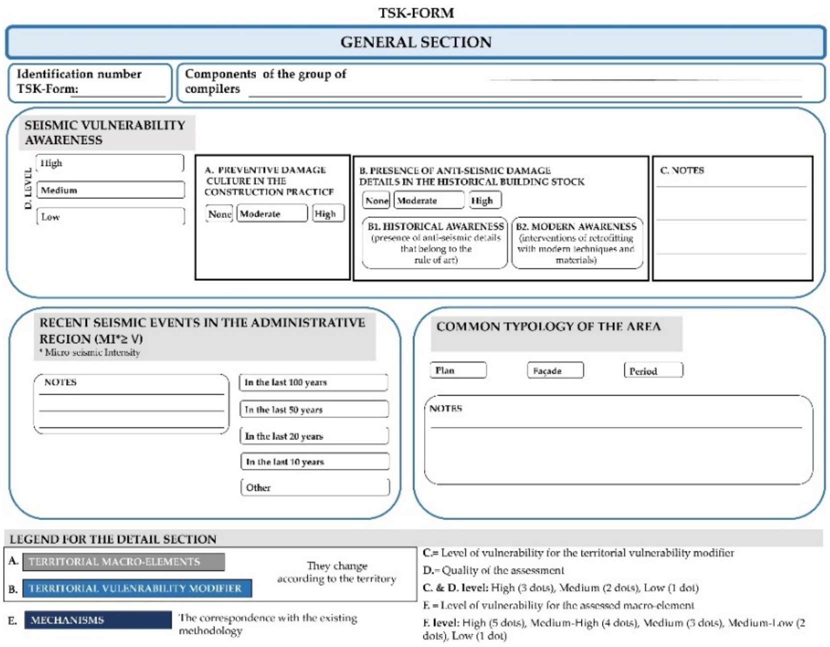


Figure 6- 3 General section of the TSK-Form. This is pre-filled by the local offices.

- *A detail section* (Figure 6- 4). This section is compiled by the team of experts commissioned by the local office, and it passes through two steps:

STEP 1. The elaboration and adaptation of the form based on a first screening of the churches present in the territory to customize the form to the local construction culture;

STEP 2. The seismic vulnerability assessment performed by a team of experts that, according to the adapted form, lead to a common level of vulnerability for each identified territorial macro-element. The section of the TSK-Form is composed of three principal columns (Figure 6- 4):

A. *The territorial macro-elements*, that are the macro-elements identified during 'step 1' and that are subject to changes from one territory to another, as the form adapted to Québec will show (Section 7.5.1). From the Italian areas and case studies investigated in the previous chapters, the TSK-Form, which in Section 6.5 will be applied to San Bartolomeo church, presents seven territorial macro-elements, that is:

1. *Façade;*
2. *Other walls*;
3. *Bell tower;*
4. *Colonnade*;
5. *Arches*;
6. *Vaults*;
7. *Dome*.

B. *Territorial vulnerability modifier* expressed in a series of questions that guide the compiler during the assessment. They are the result of literature analysis, damage assessment surveys, archive consultations, and other similar actions that enable to understand the structural behaviour of the churches. In this column of the form, the questions lead to express a level of vulnerability for that specific identified territorial vulnerability modifier, corresponding to the box indicated as '**C.**' in the form. The level could be high (3 dots), medium (2 dots), or low (1 dot). At the same time, the information derived from such assessment is weighted based on the level of accuracy of the judgement, corresponding to the box indicated as '**D.**' in the form. Depending on the possibility to get the required information during the inspection the level of accuracy can shift from *high* (3 dots), *medium* (2 dots), *low* (1 dot). The D-section was inserted also in the perspective of a statistical study.

E. *Mechanisms,* this column of the form indicates the mechanisms that could develop. At the actual state of the research, in the Italian form, exemplified in section 6.5, this column refers to the list of 28 mechanism used today, but a future implementation of this section more specific to the territory is planned, as it is performed here for the case of the néo-roman typology form (Section 7.5.1).

F. *Level of vulnerability for the identified macro-element.* This section is devoted to the final level of vulnerability, which summarizes and consolidates the results of the partial levels from the territorial vulnerability modifiers. Establishing specific criteria for the classification scale is a task for the Authorities involved and is not detailed here. However, as general indication, while the partial level may be given by three simple levels (high-5 dots, medium-3 dots, low-1 dot), the final grade may be better expressed, including a fourth and fifth ones, the medium-high (4 dots) and medium-low (2 dots). This is required by the necessity to introduce intermediate levels that depend on the frequency of the partial grades.

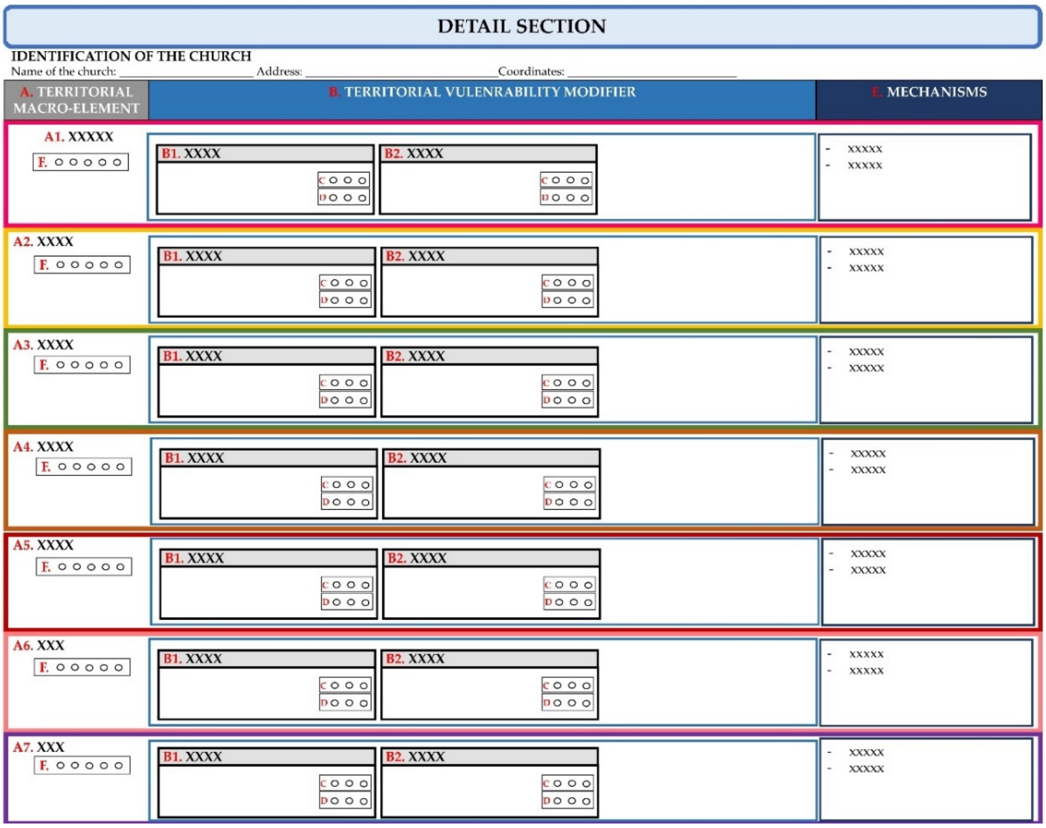


Figure 6- 4 Detail section of the TSK-Form.

6.5 AN EXAMPLE OF APPLICATION OF THE TSK-FORM TO AN ITALIAN CHURCH CASE

An example of application of the TSK-Form is presented in this section for San Bartolomeo church in Quistello (MN) and it can be consulted in Appendix III. In this simulation, the team of experts, commissioned by a local authority to perform the survey, would assess the seismic vulnerability of the church applying the territorial knowledge approach proposed in this research. The assessment is performed on the state of the church before the 2012 Pianura Padana Emiliana earthquake.

The general section is pre-filled by the local offices. It reports the moderate level of seismic awareness that, in the local building stock, is expressed by a moderate presence of seismic provision details. In the same general section, the most recent important seismic event previous to 2012 is reported, providing indications on existing cracks or interventions that could be found during the structural assessment or archive consultation.

The detail section focuses on the information of San Bartolomeo church. Thanks to its flexibility, specificities important for interpreting the structural behaviour may be collected. In particular, the territorial macro-elements allow observing what territorial vulnerability modifiers affect the most the final level of vulnerability (high (5 dots), medium-high (4 dots), medium (3 dots), medium-low (2 dots), low (1 dot). A list of seven territorial macro-elements have been identified: *Façade, Other walls, bell tower, Colonnade, Arches, Vaults, and Dome*. Each of them has a column devoted to the territorial vulnerability modifiers that allow to express the following considerations:

- The façade is assessed with a medium-high level of vulnerability; indeed, it was one of the most affected macro-elements of the church after the 2012 Pianura Emiliana Padana earthquake.
- The gable is, here, included in the territorial vulnerability modifier of the façade. Indeed, in this territory, the baroque church typology is largely common and shows a typical soaring gable, highly vulnerable to seismic actions. In San Bartolomeo church, the possibility to report such information has shown to reflect what occurred after the 2012 post-earthquake damage survey. Moreover, the presence of the cornice between the second order of the façade and the gable, assessed thanks to the form, has shown to be a critical part for activating the development of a horizontal hinge after the first shock occurred on May 20. In this regard, the configuration and the technique of construction are determinant (Al Shawa et al. 2019; Tateo and Sferrazza Papa 2019). The formation of the hinge, for example, differs, in the rectangular façade, typical of Abruzzo. In this case, it usually occurs with a diagonal orientation in correspondence of the lateral extremities of the tympanum that are not anchored with the roof because they are soaring. Collecting such specificity, which is a result of the construction culture of the territory where the church is located, provides the following positive effects: at the building scale it allows to put in evidence an aspect that affects the vulnerability of the assessed church; and, at the territorial scale, it provides indications on a specificity that, regionally influences the vulnerability.
- The presence of the territorial macro-element ‘Colonnade’ allows to report the presence of the entablature. From the sample of churches, it results to be a

common aspect in this territory. The post-earthquake damage has shown shear cracks in this portion of the structure due to the longitudinal response of the nave. When this architectural element is not present the above system of the vaults is directly affected.

- The presence of the territorial macro-element 'Colonnade' allows to report the presence of thin brick layer vaults ('volte in folio'), that are a common construction technique for this territory, but highly vulnerable to seismic actions. The high vulnerability induced by the thin brick vaults that may be expected in this vault typology has been clearly confirmed in damage recognition campaigns (e.g. Parisi et al. 2018). In the case of San Bartolomeo church, a series of thin brick vaults characterizes the interior of the church with a typical configuration of this typology, that is, they are consecutive one to the other, separated by arches. This aspect has shown to reduce the vulnerability of the church. Indeed, the damage results isolated and does not extend beyond the two arches limiting the vault (Figure 6- 5).

a. **b.** **c.**

Figure 6- 5 Damage on the thin brick vaults of San Bartolomeo church in consequence of the 2012 Pianura Padana Emiliana earthquake: a. partial collapse of the vault adjacent to the façade and part of the arch after the 1st strong shock. b. total collapse of the vault adjacent to the façade, the arch, and the second vault. c. delimitation of the damage to the vaults due to the presence of the arch. Some shear damage is present in the dome.

The use of the TSK-Form allows to report such aspect and to assign a level of vulnerability for this territorial modifier as medium because the high vulnerability due to the technique is reduced by its configuration. At the same time the possibility to report the presence of a directly adjacent vault to the façade without an arch makes this first vault of the system more vulnerable than the others, as the post-earthquake damage has shown. Moreover, the possibility to report the boundary conditions highlights the presence of close soaring elements that, collapsing, could cause damage to this vault.

If compared with the occurred damage, better described in Section 4.3.1, the TSK-Form is capable to report specific aspects that contribute to create awareness of elements of vulnerability for San Bartolomeo church. These same elements appear in other churches of the area, in which a less detailed analysis was performed. The acquisition of data seems apt to define the characteristics of the local vulnerability, useful for damage prevention strategies.

CHAPTER 7

MONTREAL, QUÉBEC: THE METHODOLOGY IN A DIFFERENT TERRITORIAL ENVIRONMENT

This section presents a different geographical area, characterized by different seismicity and construction culture. These distinct characteristics limit or prevent the application of the methodology, currently used in Italy, which is based on the recognition of a predefined series of failure mechanisms that may not fit the case of a different context. Consequently, from the observation of the territorial specificities, new mechanisms, connected to the church typologies of this geographical area, could be considered. A previous work proposed an inventory of 109 churches on Montreal Island. Some of these churches have been surveyed and classified into four typological categories to identify territorial specificities related to the typology. The church of St. Joseph was selected as a case study for one of the identified categories: the néo-roman. The seismic vulnerability assessment is first performed using the Italian methodology to identify kinematic mechanisms among the 28 proposed. Then, the territorial knowledge approach proposed in this research is used to overcome the difficulties in interpreting the behavior of the façade and bell tower macro-elements. Thereby, the methodology is tested in a different context by applying the TSK-Form to St. Joseph church.

This section is devoted to the exportation and testing of the methodology in the province of Québec, in Eastern Canada. The Island of Montreal is located in the southwestern part of the province, at the confluence of the Saint Lawrence and Ottawa rivers. It regroups 15 municipalities, the largest being the city of Montreal with a population of 1 704 694, for a total population of 1 942 044 (Ville de Montreal 2016). The city of Montreal is among the cities with the highest density of buildings in Canada and is close to the seismic Western Québec zone exposed to a moderate seismicity. These aspects contribute to put the unreinforced masonry churches highly at risk; 109 churches were inventoried for the Catholic Diocese of Montreal in Montreal Island (Youance 2009). Their cultural heritage value is recognized, and a new trend in designing rehabilitation projects, inside these historical buildings, tends to underestimate their seismic vulnerability. These are the reasons that have motivated a collaborative research project between Politecnico di Milano and École de Technologie Supérieure in Montreal. This was carried out with the intention to transfer and verify the methodology in Québec, considering the territorial specificities proper to this geographical area.

This chapter validates the methodology, clarifying the importance and originality to refer and adapt the original methodology based on 28 kinematic mechanisms to the new territory.

The attention was focused on the territorial specificities identifiable in Eastern Canada. These specificities were identified during the survey of fourteen selected churches, some of which examined in detail recurring also to interviews with people from the local community, and to Diocese Archive consultation. One of the difficulties encountered in collecting data was the absence of documentation on most of the churches, in some cases due to past fires that destroyed most of the archive documents. The general approach was to understand the local construction culture, contextualizing the church building in its territory and local construction tradition. From the inventory of the Montreal churches, four principal typologies were identified according to the façade configuration, and some considerations related to these typologies were elaborated. Among the four typologies, the néo-roman was selected and studied more in detail recurring to a case study, St. Joseph church. The common characteristics of the néo-roman typology is the position of the bell tower in the middle of the façade and the combined use of masonry and timber for this architectural element. Section 7.4 of this thesis is dedicated to the néo-roman typology and a master thesis from ETS has just started investigating another typology from the identified ones, the Conefroy. This is one of the open paths that this research has activated.

7.1 Territorial seismic context: seismic hazard and historical context

The city of Montreal is the most densely populated city in Canada and is exposed to a moderate seismic hazard. Consequently, this situation makes Montreal the second largest city at risk in Canada as shown in Figure 7- 1 (Adams et al. 2002).

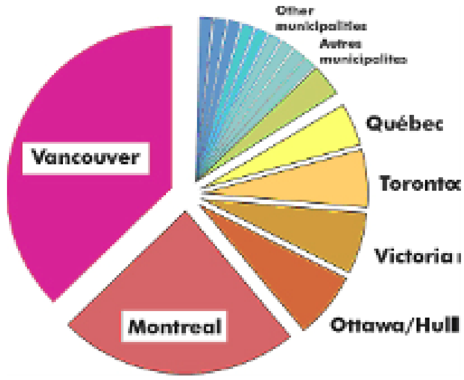


Figure 7- 1 Distribution of seismic risk in Canadian cities (Adams et al. 2002).

The understanding of the seismicity and seismic history of the territory, where the church case studies are located, is fundamental in a seismic vulnerability study. Figure 7- 2 shows the seismic hazard map of Canada. This is one of the hazard maps reporting the values of spectral acceleration for a period of 0.2 seconds for a probability of exceedance of 2% in 50 years, for firm ground condition (Class C soil), in application of the most recent Canadian building code, the CNBC 2015 (IRC-CNRC 2015).

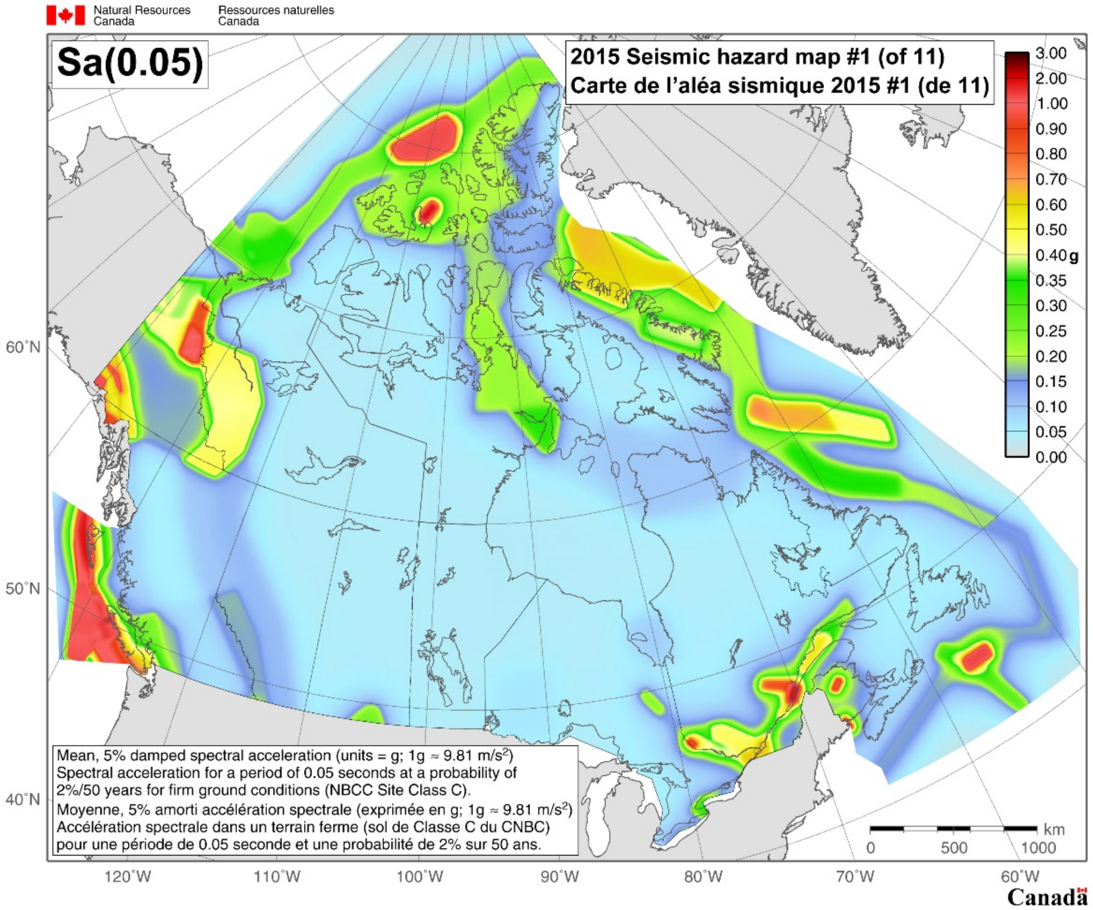


Figure 7- 2 The Canadian Hazard map for spectral acceleration at 0.5 s for a probability of exceedance of 2% in 50 years (Natural Resources Canada 2018).

The Pacific coast of Western Canada, as a segment of the Ring of fire, is exposed to a high seismic hazard, while in Eastern Canada, the seismic hazard is generally moderate. This phenomenon is still under investigation by the experts. Indeed, the province of Québec is on a large stable tectonic plate of the North American Continent. Many hypotheses have been formulated to explain the phenomenon. A generally accepted one, at the actual state of knowledge, is that the seismicity is due to the reactivation of a system of rift valley faults along the Saint-Laurent and Ottawa river. Filiatrault, in his book (1996), reports that from the comparison of the seismic events between east and west of Canada it emerges that the return period for earthquakes with $I_{mm} \geq$ VII (I_{mm}) is three times higher in the west coast of Canada than in the east (Filiatrault 1996). Other information on the different seismic characterization between east and west can be found in the Canadian Government official website on the seismic hazard (Natural Resources Canada 2018):

- in the East coast, << Each year, approximately 450 earthquakes occur in eastern Canada. Of this number, perhaps four will exceed magnitude 4, thirty will exceed magnitude 3, and about twenty-five events will be reported felt. A decade will, on average, include three events greater than magnitude 5. A magnitude 3 event is sufficiently strong to be felt in the immediate area, and a magnitude 5 event is generally the threshold of damage >>;

- in the West coast, << more than 100 earthquakes of magnitude 5 or greater (large enough to cause damage had they been closer to land) have occurred during the past 70 years>>.

Moreover, inserting the coordinates of two big cities, Montreal (45° 30' 31.9968'' N and 73° 33' 42.0048''W) in the East and Vancouver (49° 14' 46.6512'' N and 123° 6' 58.4136'') in the West, it is possible to compare the spectral acceleration S(0.3) and PGA for a probability of exceedance of 10% every 50 years, respectively equal to 0.140 and a PGA of 0.117g for Montreal and 0.544 and a PGA of 0.233g for Vancouver.

In the west the seismic waves are attenuated after 100 km from the epicenter, while in Québec attenuation of seismic waves is low due to the geology and soil. The seismic episode of Saguenay in 1988 is a clear demonstration of this local soil characteristic. Indeed, the shock was felt in the south direction down to Washington, D.C. (USA) and to the west, up to Thunder Bay (Ontario) more than one thousand km away (1380 km and 1720 km respectively) and damage was observed in North Montreal, 350 km from the epicentre (Nollet et al. 2013). This seismic episode and the consequent damage were deeply investigated (Bruneau and Lamontagne 1994; Mitchell et al. 1990; Paultre et al. 1993). Moreover, in the west the mechanism of tectonic fracture is influenced from the California area and the fracture of the faults arrives up to the surface. On the contrary, in the east, no earthquake caused superficial fracture. This aspect has to be considered when consulting data from the hazard map. In the case of the west, data are based on geological surveyed data, while for the east, the information are still not uniform and sometimes incomplete. Another important aspect to be considered is the focal depth of the seismic episode. In both cases the common focal depth is between 5 and 15 km, even if the phenomenon of subsidence in the west of Canada could give origin to earthquakes with a focal depth deeper than 15 km (Filiatrault 1996).

The seismic risk is further increased in old sectors of the city with a high concentration of unreinforced masonry buildings. The URM churches are highly vulnerable and, additionally, the cultural value of such building stock, with the oldest assets dating back to the 17th century, increases the associated seismic risk. Figure 7- 3 shows the relative seismicity of the Québec province. The most populated cities, such as Québec City and Montreal, are in the south of the province.

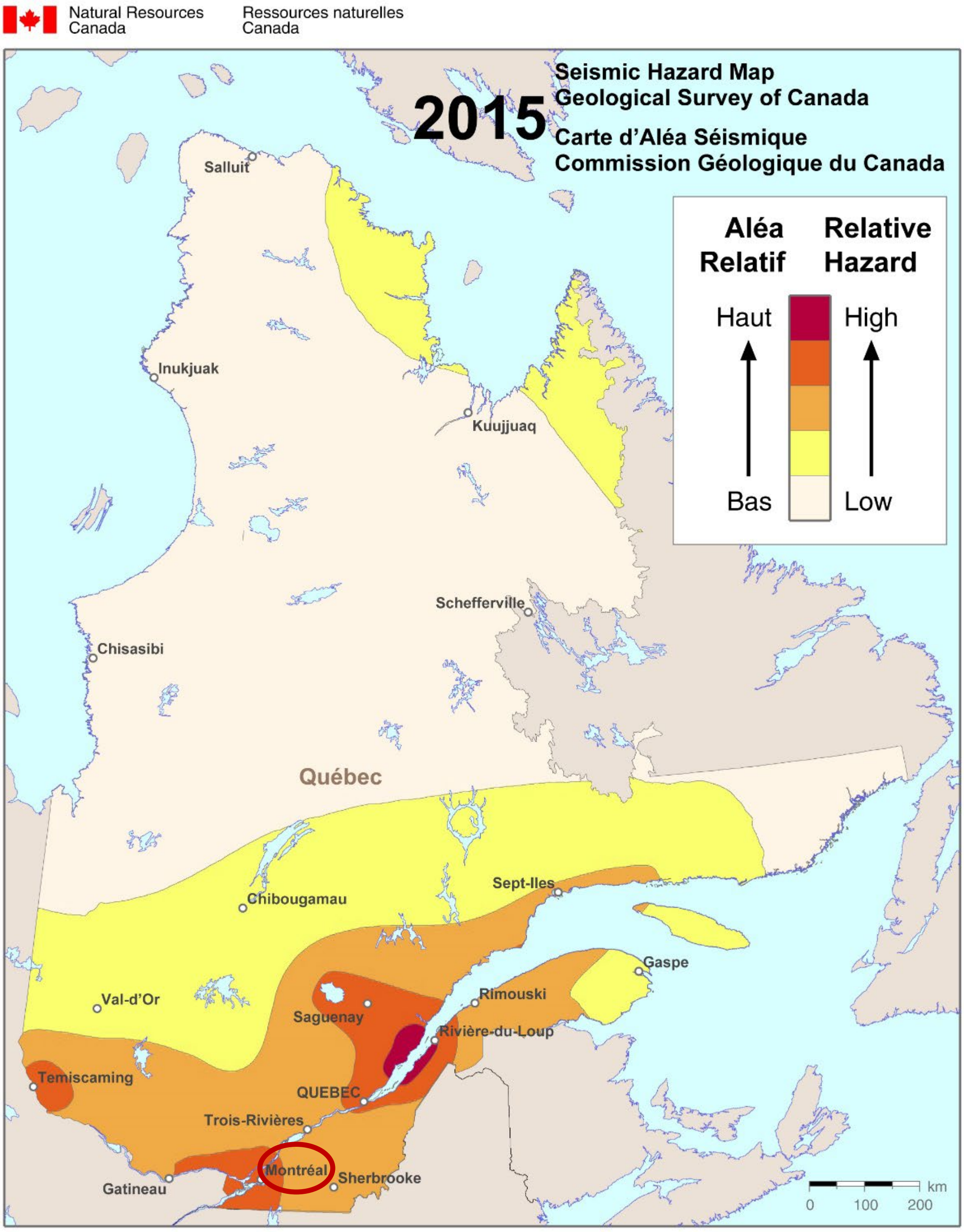


Figure 7- 3 The Seismic hazard map of Québec from the Canadian Building Code. The city of Montreal is pointed out with a red circle (Natural Resources Canada 2018).

Three areas are the most seismic of the province (Figure 7- 4): the west of Québec (''ouest du Québec'' in the map, including Montreal), Charlevoix, and Bas-Saint-Laurent.

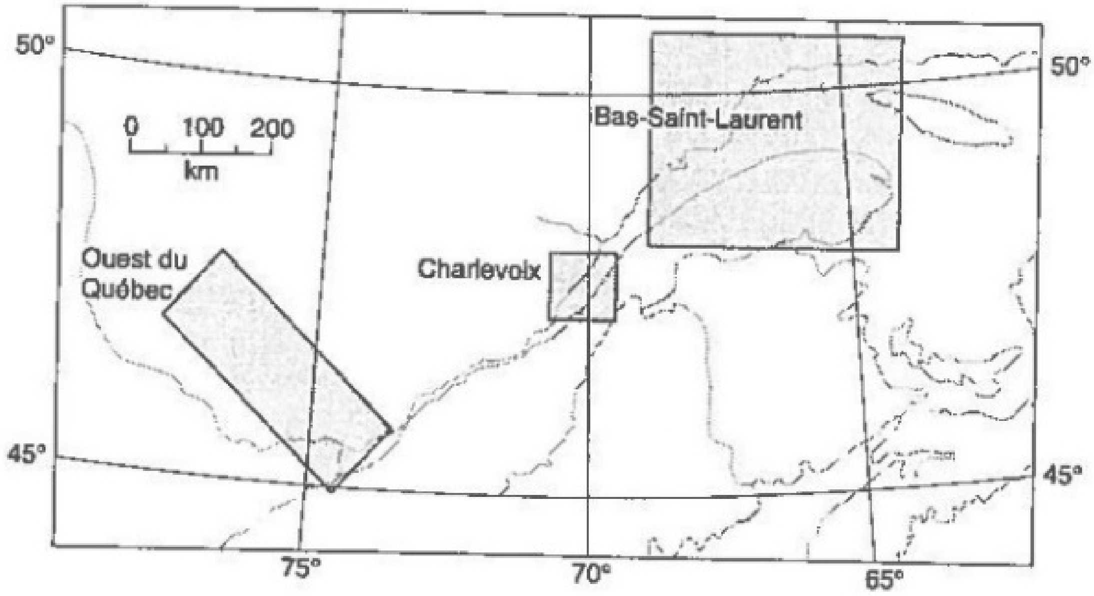


Figure 7- 4 Principal seismic areas of east Canada: the west Québec (in the map Ouest du Québec), Charlevoix, and Bas Saint Laurent. Map source, adapted from (Adams and Basham 1989).

7.1.1 Considerations on the amplification effects

In several seismic events, local site effects were observed to provoke more damage, especially in correspondence with soft sediments (Hussien and Karray 2015). According to Lamontagne (2008), on the occasion of the 1663 earthquake, major damage was concentrated in soft sediments and sand areas located 150 km from the epicenter and 300 km in case of clay soil.

In the Canadian Building Code 2015, sites are classified according to the soil composition and the average shear wave velocity in the top 30 meters V_{30}. Site classes A and B indicate hard rock, C soft rock, while D is stiff soil and E is soft soil such as soft clay. Table 7- 1, extracted from the Canadian Building Code 2015 (IRC-CNRC 2015), and available in English version in Finn and Wightman (2003), presents the seismic site classification that in turn is used to define the amplification factor reflecting the influence that the characteristics of the soils have on the transmission of the seismic waves.

Table 7- 1 Seismic site classification as a function of the shear wave velocity. Tableau 4.1.8.4.-A in (Finn and Wightman 2003).

Site class	Site class-Name and generic description	Site class_Definitions
A	Hard Rock	$\vec{V}_{30} > 1500$ m/s
B	Rock	$760 < \vec{V}_{30} \leq 1500$ m/s
C	Very dense soil soft rock	$360 < \vec{V}_{30} \leq 760$ m/s, $\vec{N} > 50$, or $\vec{S}_u >$ 100kPa
D	Stiff soil	$180 < \vec{V}_{30} \leq 360$ m/s, $15 \leq \vec{N} \leq 50$, or $50 \leq \vec{S}_u > 100$kPa
E	Soil profile with soft clay	$\vec{V}_{30} < 180$ m/s; plasticity index PI > 20, water content w > 4%, and $\vec{S}_u < 25$ kPa
F	Site-specific geotechnical investigations and dynamic site response analyses: (*i*) soils vulnerable to potential failure or collapse under seismic loading (liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, etc.); (*ii*) peats and (or) highly organic clays (H>3m of peat and (or) highly organic clay, where His thickness of soil); (*iii*) very high plasticity clays (H>8m with PI>75); (*iv*) very thick "soft – medium-stuff clays" (H>36m)	

Figure 7- 5 shows the map of Montreal Island with the seismic site categorization, as defined in the National building code of Canada 2015, and reproduced from the work of (Chouinard et al. 2012). The 14 visited churches are localized on the map.

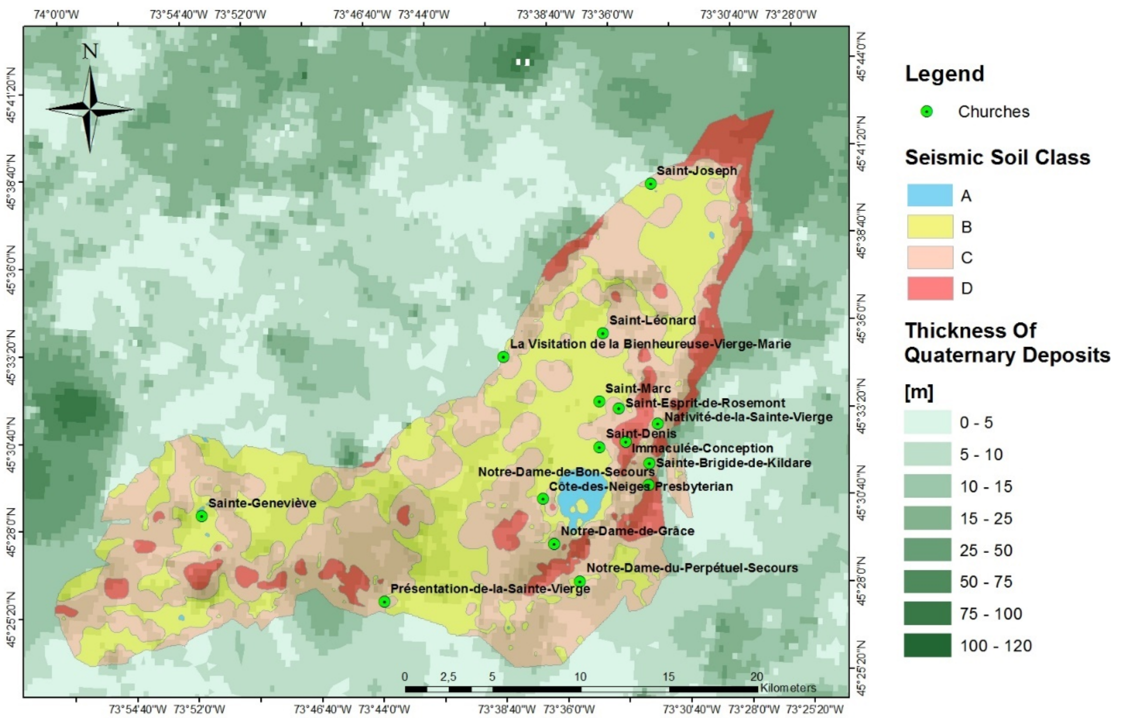


Figure 7- 5 Map of Montreal Island and location of the visited churches with the type of soil.

Among the church sample, only Notre Dame de Bon Secours and Immaculée Conception are on site of class D, while Sainte Geneviève is on a site of class B, all the others are sites of class C. Natural Resource of Canada provides a spectral acceleration referring to Site Class C. If the site differs from class C, according to the existing code (IRC - CNRC 2015, 4.1.8.1-B), an amplification factor *(Fs)* should be considered.

7.1.2 HISTORICAL DAMAGE ON CHURCHES IN QUÉBEC

Past earthquakes have caused damage to churches in Québec. Table 7- 2 lists the major earthquakes, which struck the province of Québec from the 17th to the 21st century together with a summary of the information on the damage to churches. The most damaging earthquakes occurred in the Charlevoix region in 1791, 1860, 1870, and 1925. In Figure 7- 4, this area is clearly identified. The damage reported in Table 7- 2 has to be read bearing in mind that before the end of the 17th century most buildings were built in timber. It is only after major fires that occurred in 1682 and in 1720, respectively in Québec and in Montreal, that masonry construction was enforced inside the fortification walls of the cities by Intendant Dupuy of Nouvelle France (Youance 2009).

Gouin (2001) is a precious reference documenting the impact of earthquakes before the 1925 quake. Indeed, it is only at the beginning of the 20th century, that information on seismic events could rely on recorded seismographs data, rather than conventional descriptions of the events. In any case, these observations more or less detailed, according to the case, constituted precious information on the damage caused by earthquakes. An extract from Gouin (2001) describing damage to a church during the 1663 earthquake is reported here:

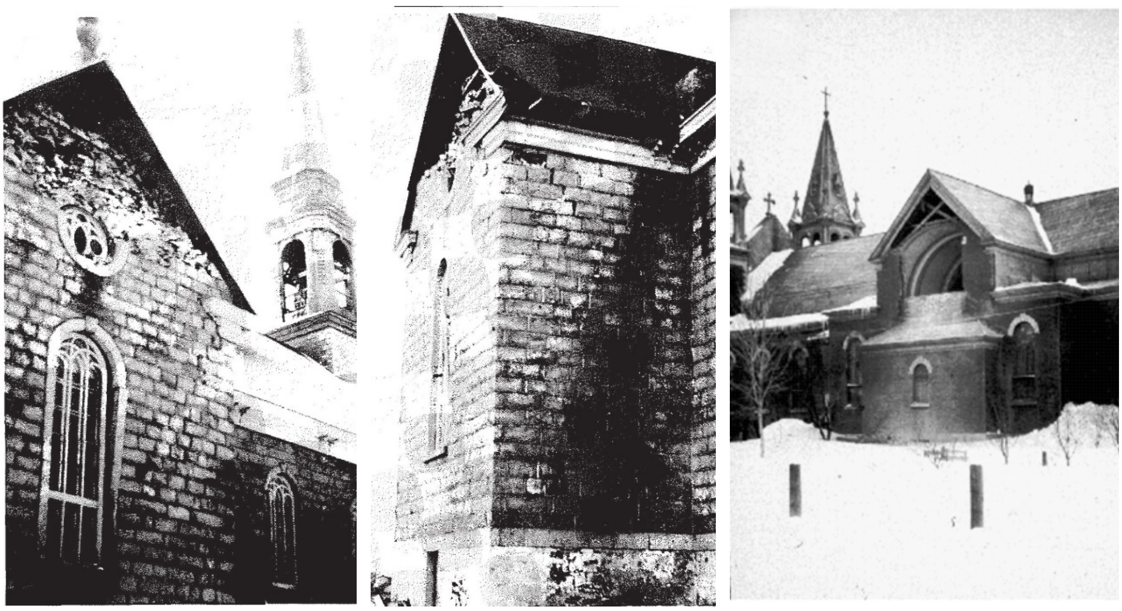
«L'église a beaucoup souffert: une partie de son portail s'est écroulée emportant un morceau de la voûte et le reste des murs est tellement lézardé qu'il est douteux qu'on puisse le réparer» (Gouin 2001, p. 268).

Table 7- 2 Principal occurred earthquakes and the registered damage on churches (Nollet et al. 2013). Note: the chimneys are common in the churches because of the used heating systems.

Year	Magnitude M_w (*Estimate)	Region	Reported damages to unreinforced masonry structures and elements
1663	7*	Charlevoix-Kamouraska	Nonstructural damage to churches / Collapse of chimneys
1732	5.8*	Montréal	Bending of bell towers / Light damage to houses / Failure of chimneys
1791	6*	Charlevoix-Kamouraska	Damage to 3 churches
1860	6*		Failure of one bell tower and wall cracking
1870	6.5*		Severe damage to 2 churches: Collapse of the portal and part of the vault, cracking of walls
1925	6.2		Collapse of one church (out of plane failure of lateral walls and roof collapse) / Severe damage to 2 churches: Falling of blocks of the bell tower, out of plane failure of unreinforced walls, shear cracking, / Collapse of chimneys / Severe damage to masonry houses
1935	6.1	Témiscamingue	Damage to 80% of chimneys and masonry walls
1988	5.9	Saguenay	In plane shear failure of unreinforced masonry walls an infill and cracking at opening corners / Out of plane failure of unattached partition walls and masonry claddings / Damage to churches (out of plane failure of façade) / Cracking of foundation masonry blocks / Damage to chimneys
2010	5.0	Val des Bois	Damage to chimneys and out of plane failure of a church gable (See Annex II for more details)

The 1663 earthquake was the strongest one that affected the region. It caused soil sliding in Charlevoix, Saguenay, Côte Nord and Mauricie; nevertheless, the reported damage is not equally impressing. This aspect is due to the low density of population and reduced height of the buildings, mostly made of timber.

Compared to the Italian abundant damage documentation for churches, the few documents found are precious for understanding the structural behavior of this building typology in relation with the specificities of the Canadian context. Here, few known pieces of information on three damaged churches, from different events, are reported: the Rivière - Ouelle and the Saint-Marc de Shawinigan churches, and the church of Notre Dame de la Visitation de Gracefield. In the first case, the church was built in 1872 on a clay soil. After the 1925 earthquake (*Mw* 6.2), some damage was registered. The overturning of the gable of the transept façade was observed, due to the lack of connection between the roof and the wall, and shear cracks in the walls and displacement of stone units in a portion of the walls of the bell tower are reported as well (Figure 7- 6a,Figure 7- 6b). Moreover, it is known that at the same location a previous church was damaged by the 1860 and the 1870 earthquakes, with estimated magnitude of 6.0 and 6.5, respectively. This church was demolished and replaced in 1872. The Saint-Marc church was also affected by the 1925 earthquake, even if the city of Shawinigan was located 250 km away from the epicenter. As in the previous example, the overturning of the gable was documented and the bell tower and the external leaf of the wall of the lower part of the wall overturned towards the exterior (Figure 7- 6c). Among the identified possible causes there are: the lack of maintenance of the buildings, many errors related to construction phases of the building, the vulnerability of the building, increased by previous earthquakes, and the fact that these buildings were rebuilt after fires. This was something that occurred frequently. Indeed, they are constituted by several timber elements that in case of fire, sometimes originated by the heating system of the church, easily burn.



(a) **(b)** **(c)**

Figure 7- 6 The images show out-of-plane modes in consequence of the 1925 Charlevoix earthquake: **a.** The overturning of the gable and shear damage of the Rivière-Ouelle church; **b.** cracks highlighting an overturning mode of the same part of the church, lateral view (Bruneau and Lamontagne 1994 p.645); **c.** overturning of the gable of a church in Shawinigan (Bruneau and Lamontagne 1994 p.649).

In the case of Notre Dame de la Visitation in Gracefield, the church was affected by the 2010 Val de Bois earthquake with a magnitude of 5.0 (Figure 7- 7). The masonry chimney (7.62 m tall) collapsed causing damage to the roof of the sacristy. Some planks in the south entrance (sacristy area) fell. The chimney was built with bricks and with stones. Some elements of the wall in the choir area fell or were dislocated. This wall is adjacent to the chimney that collapsed. No cracks in the bell tower and church wall from the interior were observed. The inspection, executed the day after the event, was also performed at the first deck of the bell tower and the inspection of the timber structure of the roof did not point out any problem (Auge 2010). The bell tower was inspected up to the bell level and no evident problem or damage were pointed out. Some metal sheets of the steel cladding of the bell tower were deformed but no information indicates if this damage occurred during the seismic event or previously. No damage at the level of the jubé, that is, the choir above the entrance, was registered. From the exterior, some mortar joints fell on the ground. Above the windows some stones moved outside, and the mortar was cracked.

(a) (b)

Figure 7- 7 Damage to Notre Dame de la Visitation in Gracefield: **a.** portion of the roof damaged by the collapse of the chimney; **b.** Zoom on the portion of the damage roof.

7.2 Typical Church Typologies (Baillairgé, Conefroy, Néo-Roman, Italian Baroque)

Starting from an inventory of 109 churches in Montreal Island (Youance 2009), a group of churches was selected for further studies. The selection was based on the occurrence of characteristic elements, like the façade typology, the presence and location of the bell tower, the plan configuration, in order to assemble a set sufficiently representative of churches for this territory and help to trace the construction culture of the area for this building category (Figure 7- 8). Indeed, the churches on the Island of Montreal are representative of church typologies found elsewhere in the province, in cities as well as in small towns (e.g. Le Patrimoine Religieux du Québec 2018). Moreover, the understanding of the behavior of the bell tower contributes to preserve this architectural element important both at the building and at urban scale, considering that Montreal is known as the "The city of a hundred bell towers" as Mark Twain proclaimed during his first visit of the city in 1888 (Québec's National Shrines 2018).

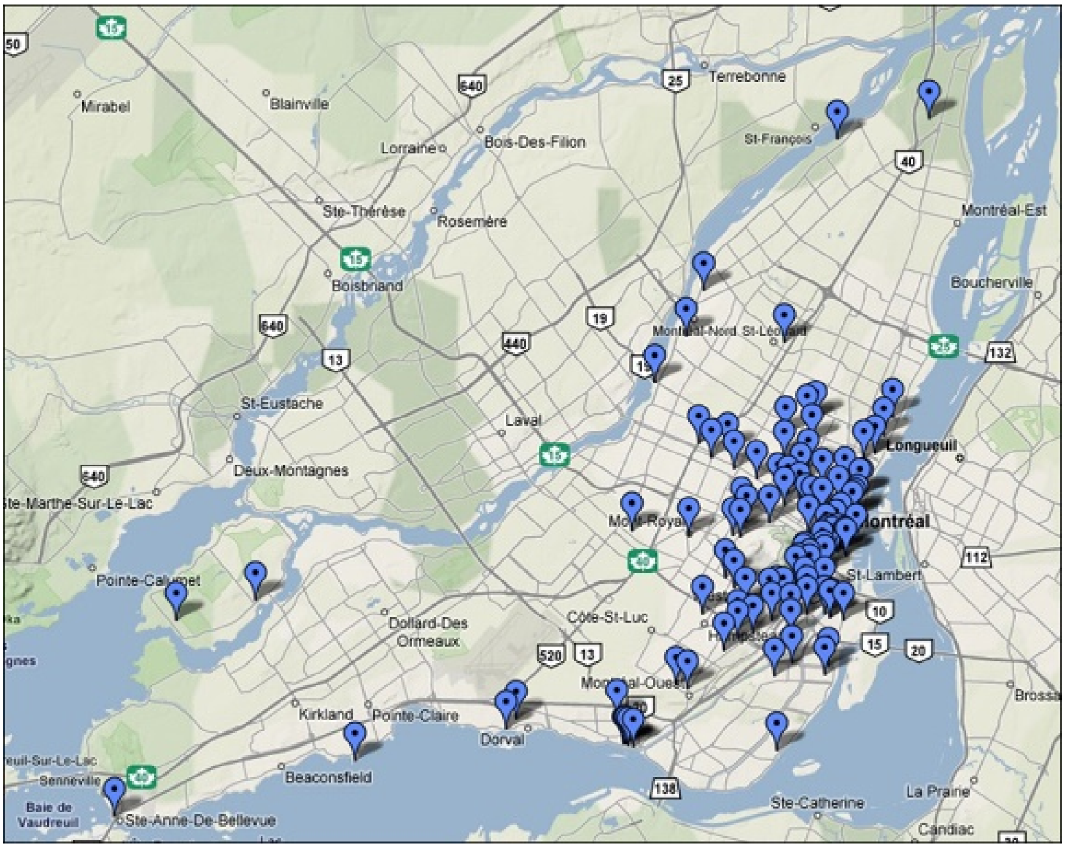


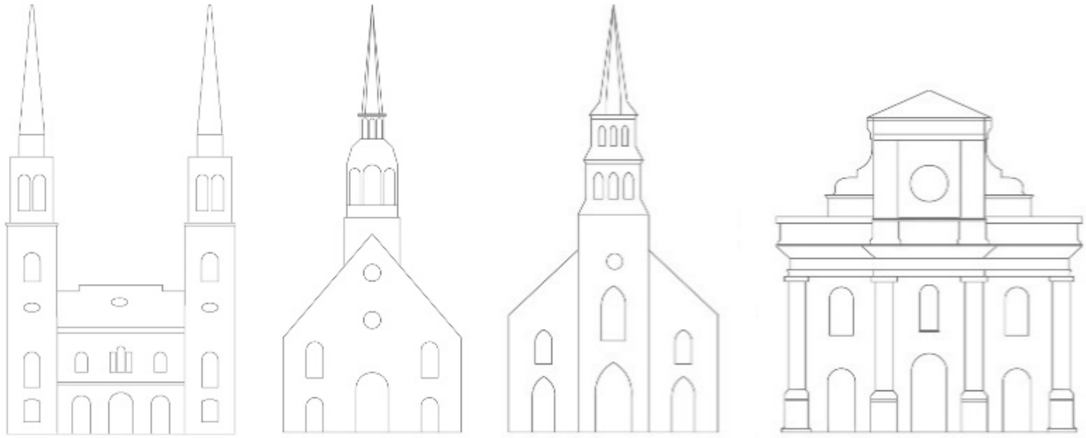
Figure 7- 8 Inventory of the 109 churches on the Island of Montreal (Youance 2009).

Figure 7- 9 indicates the set of 14 churches visited. Every church has been inspected, with the aim to identify the local construction characteristics that were expected to affect their seismic structural behavior. Based on the Italian experience, the presence of elements inducing vulnerability for these structures was checked. Some survey forms were prepared in advance in order to collect detailed and accurate information during the on-site visits. The visits included visual recognition of the state of conservation and existing cracks, materials and techniques of construction, and inspections of the roof and foundations, when they were accessible. This action allows to enter in contact with the construction practice of the territory and to find systemic structural and conservation problems that affect the structural performance of churches. Understanding the territorial specificities has constituted the base for all the analyses. Moreover, some dimensions were taken to draw plans and elevations, in some cases to elaborate 3-D models that could facilitate the understanding of the interrelation between macro-elements. The on-site visit was a crucial step of the procedure to confirm the state of conservation of the building in comparison with the information collected by Youance (2009) and at the same time to validate some hypotheses used to select the case studies.

During the on-site phase, the building was observed from the perspective of the local construction culture to suggest the most appropriate interpretations.

Figure 7- 9 The surveyed churches belonging to the sample shown in Figure 7- 8.

From the information collected from the previous inventory and the visited churches of Montreal four main church typologies were defined based on the façade geometry. Different geometries, in fact, correspond also to different internal distributions, and façades have shown the most different macro-elements in comparison with the Italian cases. The reason is the high vulnerability demonstrated by the façade macro-element in previous earthquakes. In the Italian experience, many studies focused on the structural response and the corresponding vulnerability of the church façade (Casolo et al. 2000; Casolo and Uva 2013; Casolo 2017). Working by category has allowed to observe in more detail the territorial specificities for each typology. Walking down the urban streets, most of the churches in the Diocese of Montreal can be easily associated to these principal typologies: the Baillargé (1790-1820), the Conefroy (from 1800), the Néo-roman (1880-1930), and the Italian baroque (second half of 1800). Figure 7- 10 shows the most common church façades, drawn after the surveys.



(a) (b) (c) (d)

Figure 7- 10 Façade typologies of the churches in Montreal: **a.** Baillargé, **b.** Conefroy, c. Néo-Roman, **d.** Italian Baroque.

A peculiar characteristic of these churches is the combined use of timber and masonry as construction materials. The timber is generally employed for the structure of the bell towers, the roof system, and the colonnade of the nave. On the contrary, the use of masonry is relegated to the external walls. The curtain walls are made of local stone blocks that can be roughly squared or square cut, depending on the church. They are laid with a regular pattern and, in some cases, some bond stones are present. In absence of Non-Destructive Tests (NDT), the hypothesis of a double-leaf wall was formulated for what was possible to observe at the roof level. The exterior of the church clearly shows the use of stone for the masonry walls, while the interior walls are covered with wood planks and plaster. Other specific characteristics, observed during the visit of the different churches, are reported here and organized by typology.

The *Baillargé typology*, that took the name from its Architect Thomas Baillargé (1791-1859), has a façade enclosed by two bell towers (Figure 7- 11). The façade has a symmetrical layout. At the ground floor, it generally presents a principal entrance door in the middle of the façade, and openings or niches on both sides of the entrance. At the second order of the façade, a large central opening, on top of the entrance, and other openings or niches, organized symmetrically, can be observed.

(a) **(b)**

Figure 7- 11 Two examples of churches belonging to the Baillairgé typology in Montreal: **a.** Sainte Genevieve; **b.** Visitation de la Bienheureuse Vierge Marie. More detail in Appendix IV.

In this typology, the façade and the two bell towers create a sort of diaphragm that separates the exterior from the interior of the nave with an atrium, constituting a massive structure annexed to the rest of the structure of the church. This church typology has typically three naves separated by a colonnade. Figure 7- 12 shows an example of the plan and elevation of this typology surveyed on-site for the case of Sainte Geneviève church.

The bell towers are made of masonry up to the first level of the belfry that is higher than the cornice of the gable, while the rest of the structure, which continues above, is made of timber, sometimes painted and other times covered with laminates. Figure 7- 13 shows an example of Baillargé church with its three naves.

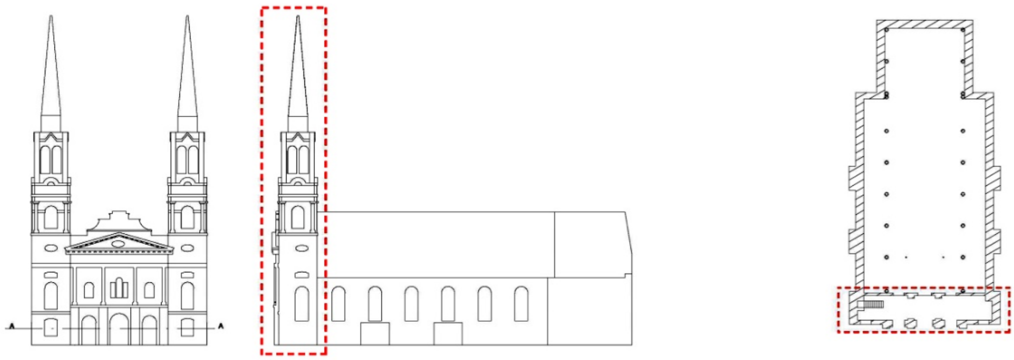
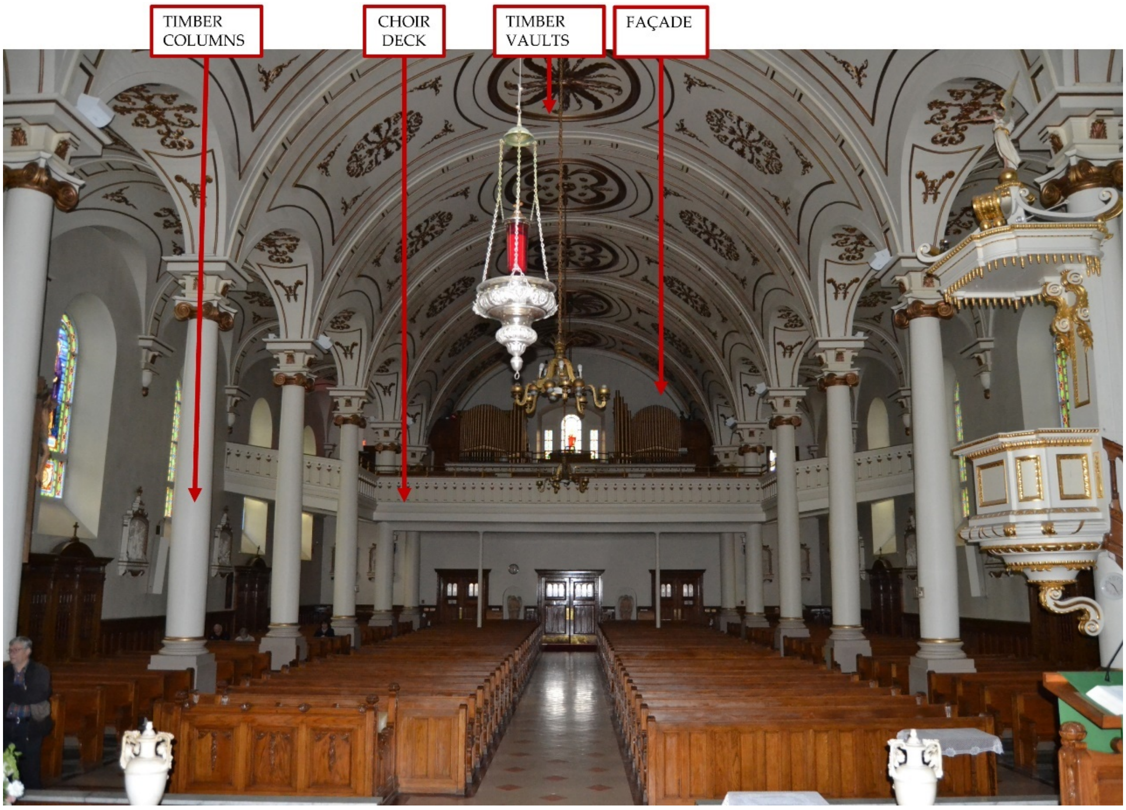


Figure 7- 12 Plan and elevation of Sainte Genevieve church.



(a)

(b)

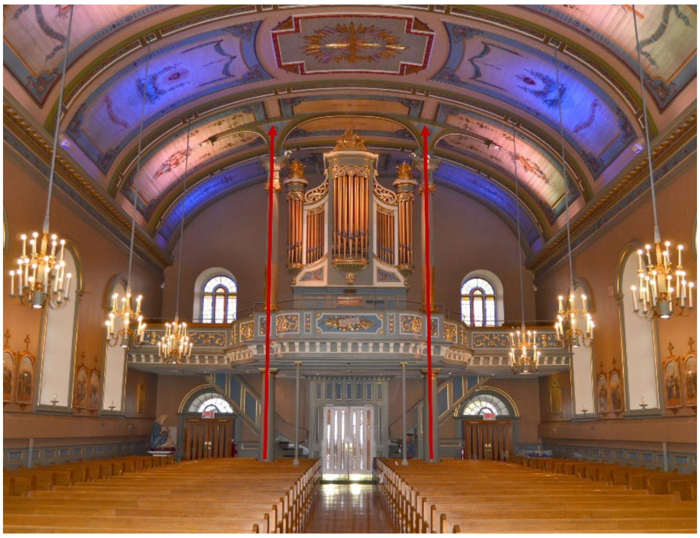
Figure 7- 13 Interior of Sainte Genevieve: **a.** View towards the façade; **b.** View towards the apse.

The *Conefroy typology*, name due to abbot Conefroy that designed the plan configuration of such typology, is characterized by a bell tower located in the middle of the façade, set back from the façade plane (Figure 7- 14). The façade is frequently characterized by a symmetrical shape in axis with the bell tower and the entrance door, which gives direct access to the nave of the church. Indeed, this church is characterized by a single nave. Two rose windows, one under the other, are aligned along the axis of symmetry of the façade. One such opening is close to the cornice, at the level of the roof, and one gives light to the nave. Moreover, two large windows are located on the left and right portion of the façade. In some cases, two doors are also present, aligned with the windows, to provide additional access to the church.

(a) **(b)**

Figure 7- 14 Two examples of churches for the Conefroy typology: a. Sainte Famille in Boucherville; b. Notre Dame de Victoire in Québec city.

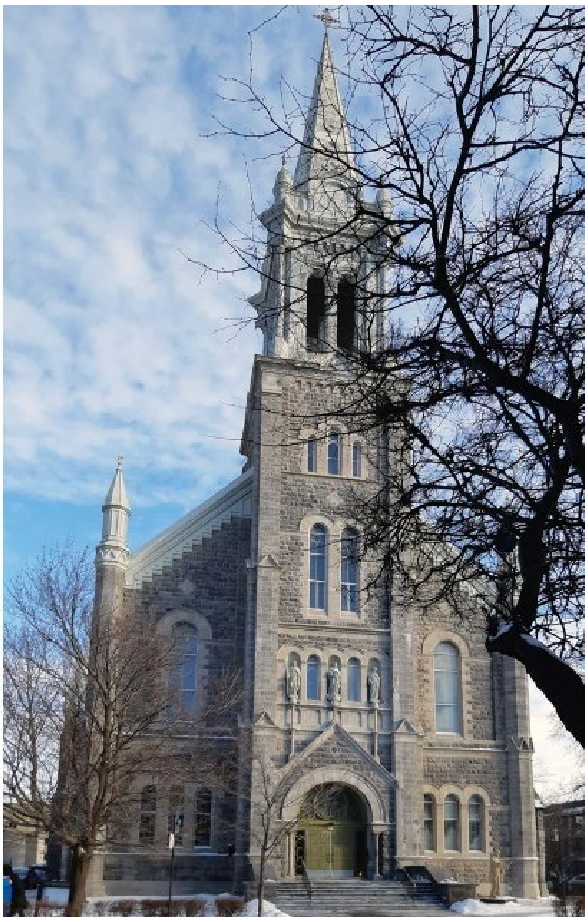
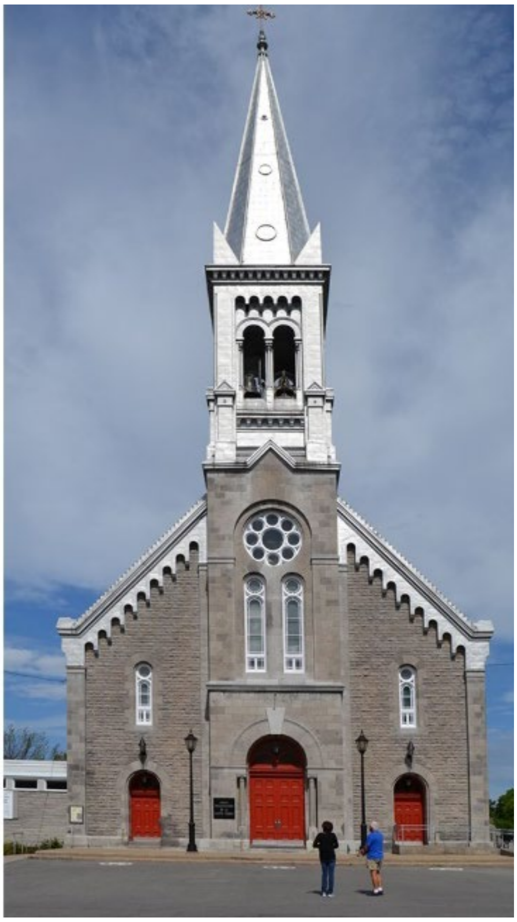
The pillars of the bell tower are well visible inside the church at the beginning of the nave (Figure 7- 15). Four pillars support the entire structure of the bell tower. They generally describe a square shape, that supports the upper part of the bell tower, and they constitute a characteristic element in this church typology. In the interior of the church, the pillars are in two pairs, one pair is close to the façade and the other penetrates the jubé level, the inner tribune over the main entrance, ending at the ground level of the church.




(a) (b)

Figure 7- 15 Pillars of the bell tower in the Conefroy typology (red arrows): **a.** Sainte Famille in Boucherville; **b.** Notre Dame de Victoire in Québec city.

The *néo-roman typology* is characterized by the position of the bell tower that, occupying the central part, interrupts the continuity of the façade and provides the characteristic configuration visible in Figure 7- 16.

(a) (b)

Figure 7- 16 Two examples of churches for the néo-roman typology in Montreal: **a.** Immaculée Conception; **b.** Présentation de la Sainte Vierge.

The detailed inspection of this typology was fundamental for the understanding of the structural system that involves the façade and the bell tower. Visiting also the roof and the bell tower, the mixed-use of timber and masonry was observed. This complex structural configuration makes this typology particularly interesting and worth being more deeply investigated (see section 7.4). Néo-roman churches have typically three naves (Figure 7- 17).

Figure 7- 17 Example of interior for the néo-roman typology: Presentation de la Sainte Vierge in Montréal.

The *Italian baroque* typology is characterized by the high slender and overhanging façade with pilasters. These elements mark the design of the façade and the volutes embellish the tympanum at each side. This church typology is composed of three naves that repeat the exterior rhythm of the façade. Most churches of this building typology have unfortunately been demolished. Figure 7- 18 shows one example of an Italian baroque church in Montreal.

Following some renovation works, the interior was transformed from the original state, keeping the three-nave distribution, but replacing the vault and arch system, typical of a baroque church, with a flat non-structural slab (Figure 7- 19). Consequently, the structural concept was deeply modified. It should be noted that in the renovation work the wood pillars have been covered by marble panels.

Figure 7- 18 Example for the Italian baroque typology: Notre Dame de Grâce in Montréal.

Figure 7- 19 Interior of Notre Dame de Grâce.

7.3 Considerations on the seismic vulnerability of the identified typologies

Walking down the streets and observing some historical masonry buildings in Montréal, one can observe some buildings with tie rods at the level of the slabs. Figure 7- 20 shows an example of building on St. Paul Street in the Old Montréal, one of the principal streets of the historical center.

Figure 7- 20 Old tie rods at the level of the slabs in both sides of this old house in the Old Montréal.

This building is just across the street from one of the visited churches, Notre Dame de Bon Secours.

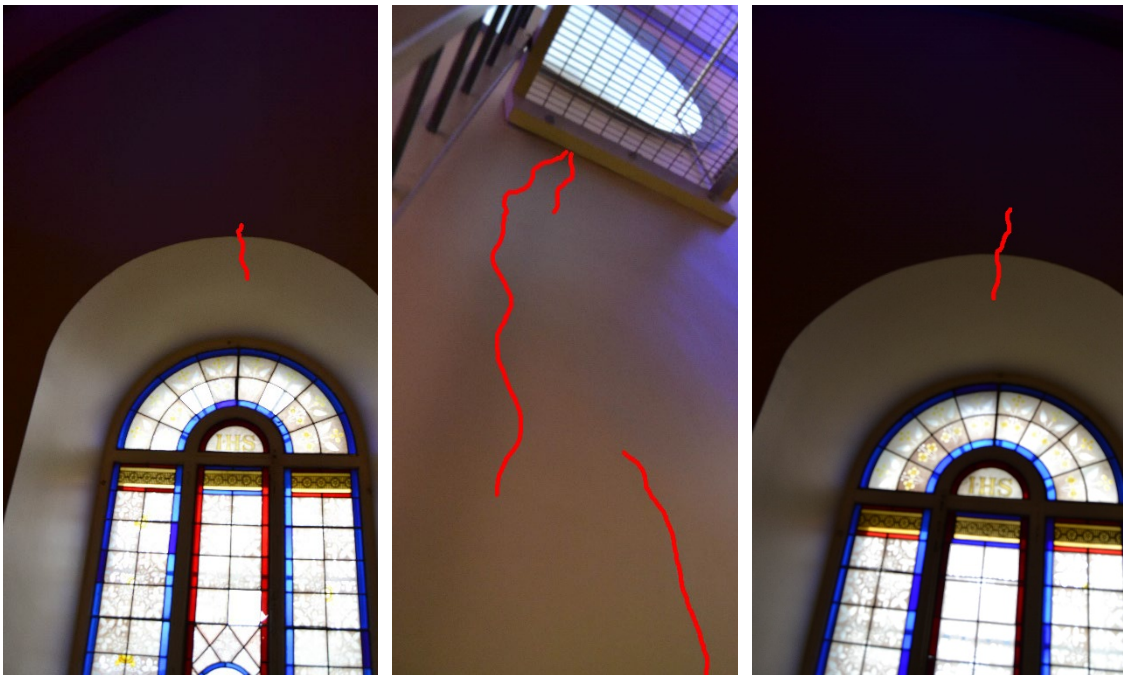
As a first step, the recommendations proposed in the Italian guidelines of Cultural Heritage for the seismic vulnerability assessment of churches (MiBAC 2011) were applied. From a first screening, the absence of tie rods in most of the churches with vaults or arches was put in evidence. The only case where tie rods were found was the church Notre Dame du Bon Secours, in the historical center of Montreal. The church is classified as ‘A' according to the regional scale of value of protection and conservation elaborated by the Council of the Religious building stock of Québec (Conseil du Patrimoine religieux du Québec 2018). Such lack of seismic provisional details motivates the statement that there is a low awareness of seismic vulnerability for URM historical buildings in this part of Canada.

As a second step, the interpretation of the church structure was performed by identifying the principal macro-elements and the associated kinematic chain mechanisms, referring to the listed 28 ones. Immediately, this action pointed out the difficulty to apply directly the procedure. Indeed, some of the specific characteristics of the different typologies identified and the construction practice of this geographical area make it difficult to interpret directly the structural behavior in case of an earthquake. Some of the considerations that have emerged, distinguished by typologies, are the following:

- *Baillargé typology (*Figure 7- 10*a)*. In this case, the façade is not directly connected with the longitudinal walls of the nave, but it is attached to the two bell towers. Moreover, the façade and the bell towers, by their configurations, define an intermediate space, structurally located before the beginning of the colonnade of the nave and the rest of the church structure (Figure 7- 12). A global numerical model could facilitate the understanding of the influence of such a massive structure directly connected with the nave. The gable, considering its position, incorporated between the two bell towers as the rest of the façade, also deserves to be studied more in detail. A local numerical model could provide an interpretation of the possible mechanism associated with this macro-element. Another characterizing aspect for this type of churches is the mixed-use of masonry and timber for the structure of the bell tower: the first is used up to a level above the façade, while the second is employed in the higher portion. The transversal response of the nave, as well as the longitudinal one of the colonnades, requires a deeper study. Indeed, the mixed-use of timber and masonry makes the structural interpretations difficult. The in-plane mechanisms from the listed 28 ones, which describe possible shear cracks in the walls, are valid here, due to the similarities with the Italian churches for the perimetral wall structure.

- *Conefroy typology (*Figure 7- 10*b)*. The particularity of this typology is the position of the bell tower and the way it is built in relation with the rest of the church structure. The columns of the bell towers do not interact with the façade structure because they are structurally independent and run parallel to the façade; however, at the roof level the longitudinal beams are embedded in the façade wall. The kinematic chain mechanisms, associated with the façade and the bell tower, have to be reconsidered in light of these specific characteristics. The behavior of the remaining parts of the structure can be easily interpreted from the other listed mechanisms. The interlocking between the walls that constitute the perimetral wall structure of this type of church, is generally well-defined with square stone blocks. However, some identified cracks that could trigger the activation of some mechanisms in case of earthquakes were observed. In particular, they were found around the central rose window (Figure 7- 21) and symmetrically in the façade plane close to the openings (Figure 7- 22). To better understand the influence of the bell tower position on the structural response of Conefroy churches, a specific study is in progress.

(a)



(b) (c) (d)

Figure 7- 21 Cracks in Sainte Famille in Boucherville: **a.** Rose window; **b.** Left opening in the façade wall at the second order; c. Below the rose window; **d.** Right opening in the façade wall at the second level.

(a)

Figure 7- 22 Cracks in the façade wall of Notre Dame de Bon Secours: **a.** complete view towards the façade wall; **b**. zoom on the crack below the left opening; **c.** zoom on the crack above the left opening.

- *Néo-roman typology (*Figure 7- 10*c).* In this case, the bell tower has a different position compared to what is typical in the Italian churches: isolated (Figure 7- 23a) or as part of the church structure on one side of the façade (Figure 7- 23b), or in the proximity of the apse (Figure 7- 23c).

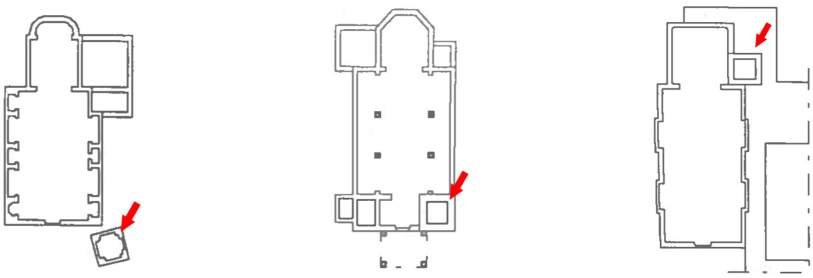
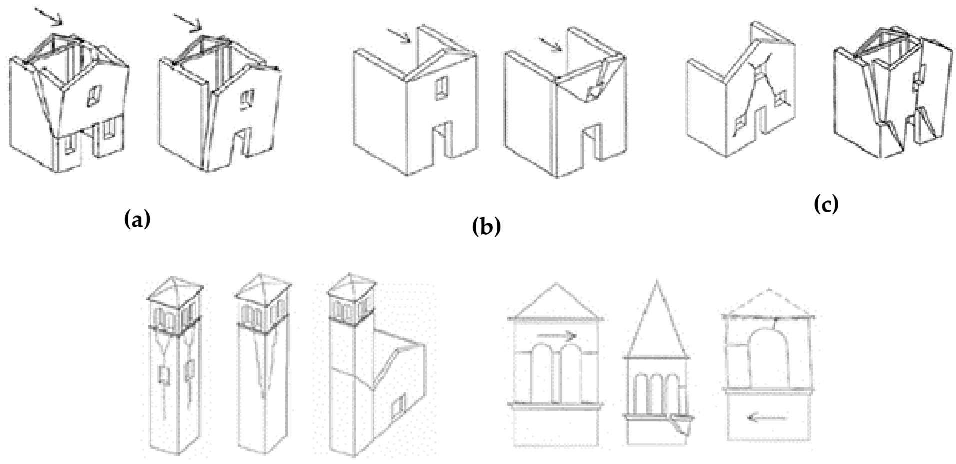


Figure 7- 23 Typical position of the bell tower in the Italian church typologies: **a.** independent from the rest of the church; **b.** on one corner of the façade; **c.** on the backside of the church included in the structure of the church.

In the néo-roman churches, the position of the bell tower, that interrupts the façade plan, and the mixed-use of timber and masonry do not make it possible to interpret directly the structural behavior according to the identified mechanisms usually associated to the macro-elements "façade" and "bell tower":

- Mechanism 1: overturning of the façade (Figure 7- 24a);
- Mechanism 2: mechanism of the upper part of the façade (Figure 7- 24b);

- Mechanism 3: in-plane mechanism of the façade (Figure 7- 24c);
- Mechanism 27: mechanism of the bell tower along the height (Figure 7- 24d);
- Mechanism 28: mechanism of the upper portion of the bell tower (Figure 7- 24e).



(a) **(b)** **(c)**

(d) **(e)**

Figure 7- 24 Mechanisms related to the macro-element façade and bell tower: **a.** Mechanism 1; **b.** Mechanism 2, **c.** Mechanism 3; **d.** Mechanism 27; **e.** Mechanism 28.

It is necessary to go beyond the rapid 28-kinematic mechanisms procedure. The consideration of other aspects of these macro-elements, such as how they are built and what is the connection between the bell tower and the façade, becomes crucial. Visual inspections of the above church examples show that the structure of the bell tower is strictly connected with the façade, the jubé and the lateral walls. Analysis of the construction materials visibly shows that the structure of the bell tower is made of timber while the supporting walls and façade are in multi-leaf stone masonry as shown in the St. Joseph case study (Section 7.4). Figure 7- 25 shows the timber structure of the bell tower observed from the interior of some churches at the level of connection with the masonry of the façade.

(a) (b) (c)

Figure 7- 25 Examples of the structure of the bell tower at the level of connection with the façade: **a.** the timber structure of the bell tower is adjacent to the façade masonry wall; **b.** the structure of the roof intersects with that of the bell tower; **c.** the timber structure of the bell tower starts inside the masonry of the façade ('pan de bois').

Moreover, in this typology, some cracks were frequently found: some symmetrical cracks were observed in the longitudinal walls of the nave in the proximity of the façade and in the façade plane above with the large windows at the second level of the façade.

Due to the local construction culture, different from the Italian one, this typology was investigated more in detail recurring to a case study (Section 7.4).

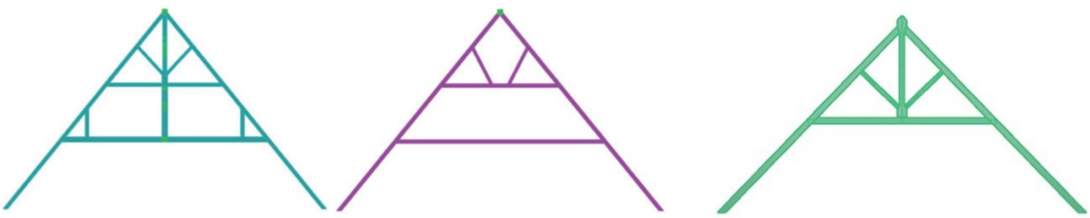
- *Italian baroque typology* (Figure 7- 10d). This is the typology most similar to the Italian cases. The soaring gable without any seismic provisional details presents a high vulnerability. However, in the case of the examined church, modifications made to the inside of the church replacing the vault and arch system by a flat slab, have modified the structural response of the church structure to lateral forces. Although only a few churches belong to this typology, it is a clear example of how modifications not related to structural needs can alter the structural conception of a historical building.

Another significant aspect related to the construction tradition of this territory is represented by the structures of the roof of these churches. In Figure 7- 26, three possible roof configurations, found during the roof surveys are shown: in the first case, the roof structure is composed by two alternated types of trusses: a principal and a secondary one (Figure 7- 26a); in the other two cases, the same type of truss is repeated unaltered along the length of the nave (Figure 7- 26b). Figure 7- 27 shows in form of simplification the explained examples.

(b)

(a) (c)

Figure 7- 26 Examples of roof structures of visited churches in Montreal Island: **a.** Sainte-Famille, **b.** Présentation de la Sainte Vierge, **c.** Saint-Joseph.



(a) (b)

Figure 7- 27 Simplification of the roof trusses shown in Figure 7- 26: a. principal (on the left) and secondary truss (on the right), as in Figure 7- 26a. Scheme's rights: Roxanne Carrier; **b.** truss shown in Figure 7- 26b,c.

Often difficult to be inspected, this part of the building plays an important role in the box-like behavior of the structure, a key element for good seismic performance (Parisi, et al. 2012; Parisi et al. 2013).

Other general considerations that interest all the typologies are the following:

- *The understanding of the structural behavior of the bell tower in the churches considering the territorial specificities.* Indeed, in Montreal island, the bell towers characterize the skyline of old historical centers, often becoming a reference point in the urban scenery, and, in earthquake-prone areas, in particular referring to the Italian experiences, this architectural element has demonstrated being extremely vulnerable to seismic actions. The long Italian seismic history provides a series of studies and documentation on their earthquake damage (e.g. Acito et al. 2014).

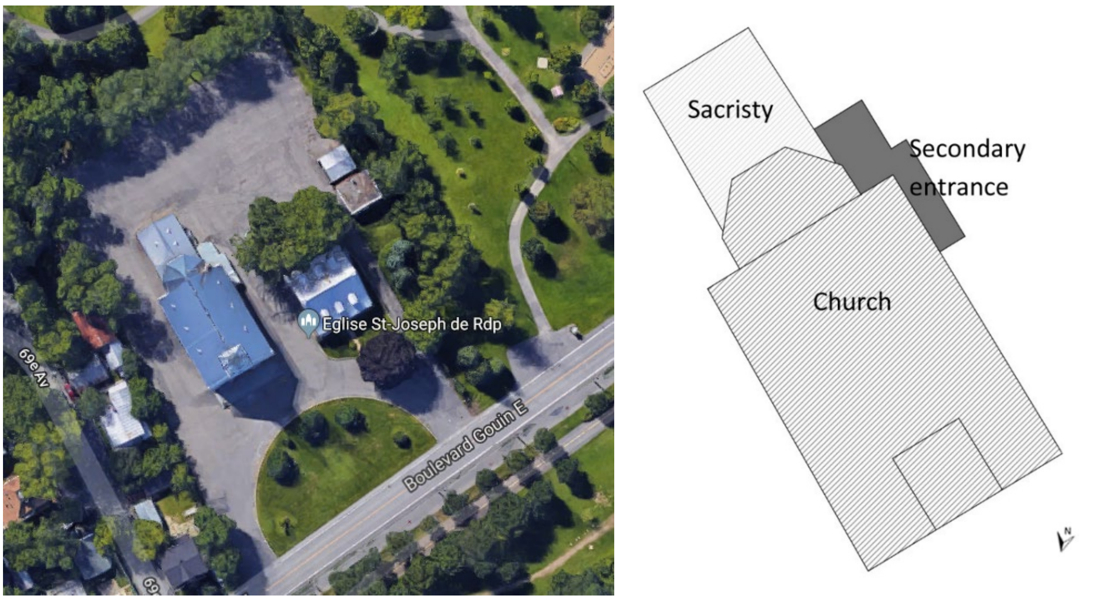
- *The severe Winter of the northern American continent.* This geographical area is exposed yearly to several cycles of freeze and thaw and outdoor temperatures can reach -30° C. These thermal cycles could cause cracking of the stone elements of the walls, leading to local structural problems. This is due to the *frost weathering*: with such a drop in temperature the water within the stone elements turns to solid-state. The increased pressure in the porosity of the stone could produce internal microcracks reducing the mechanical properties of the stone. When these molecules of ice melt, they leave empty spaces in the stone elements. Less structurally relevant but worth noting is the phenomenon of humidity frequently found in the interior of the churches due to the condensation. It is usually observed by the deterioration of the plasters, used to cover the wood plank layer on the walls. This is due to the difference in temperature between the interior (heated areas) and the exterior of the church.

7.4 The case study: St. Joseph Church

The church of Saint Joseph is an isolated church in the north of Montreal Island, in a suburb called Rivière-des-Prairies. The presbytery is aside from the church building, in a lot facing the river des Prairies (Figure 7- 28).

Figure 7- 28 St. Joseph in Prairie neighbourhood: the church and the presbytery.

In several visited churches, the building of the presbytery and that of the church were separate, although they had been built in the same period. Being two separated buildings, the church can be studied independently. Figure 7- 29 shows the church building that is composed of three parts: the church, the sacristy, and an annexed part with the function of secondary entrance. This last element will not be considered because it is a light metal structure, added for functional reasons in recent times.



(a) **(b)**

Figure 7- 29 St. Joseph church: **a.** Aerial view of the complex church and presbytery; **b.** The distinct volumes that constitute the body of the church building.

For this church, only limited information on the history and on interventions was found. This is a common problem faced during the investigation. After the demolition of a previous church at the same location, Saint Joseph church was rebuilt between 1875 and 1876. The project of the church is associated with the figure of Victor Bourgeau for the similarities with Saint-Joachim and other works designed by this same architect. In 1937, the bell tower burned because of a flash of lightning. It was rebuilt with the same material (timber) and the same technique of construction. The floor was renovated in 2000 (Youance 2009).

The way Saint Joseph is built is representative of the construction practice in the Eastern Canadian territory for churches. The combined use of two materials, stone masonry and timber, is majestically performed. The masonry is used for the construction of the wall perimeter of the church, while the timber is used for the soffit and the decorations of the interior and the structure of the roof, the bell tower, and the colonnade. An extraordinary dissimulation game between decorative and structural elements sometimes characterizes the church.

Saint Joseph church presents simple elevations (Figure 7- 30). The peculiar elements that characterize the principal elevation of the church are two buttresses, the three doors, and windows of the upper portion of the wall that break the continuity of the plane of the façade, and the bell tower, in the middle of the façade. This is a characteristic element for néo-roman churches.

(a) (b)

(c) (d)

Figure 7- 30 Principal elevations of St. Joseph church: a. The façade; b. South elevation; c. North view; d. East view.

The principal façade is externally made of squared blocks of limestone, laid in regular joints (Figure 7- 31a). The interior inspection of the roof level allowed us to observe the interior fabric of the façade wall that differs from the exterior one because it is made of roughly squared stones of different size without a regular arrangement (Figure 7- 31b). In the absence of non-destructive tests, the section of the façade wall - about 90 *cm* – was assumed constituted by two leaves, the internal (roughly squared) and external one (square cut). From the external pattern, no transversal block, that could guarantee the interlocking between leaves, is visible. On the contrary, between the different walls of the church, the interlocking is guaranteed by square blocks well defined and with an alternated pattern (Figure 7- 31c).

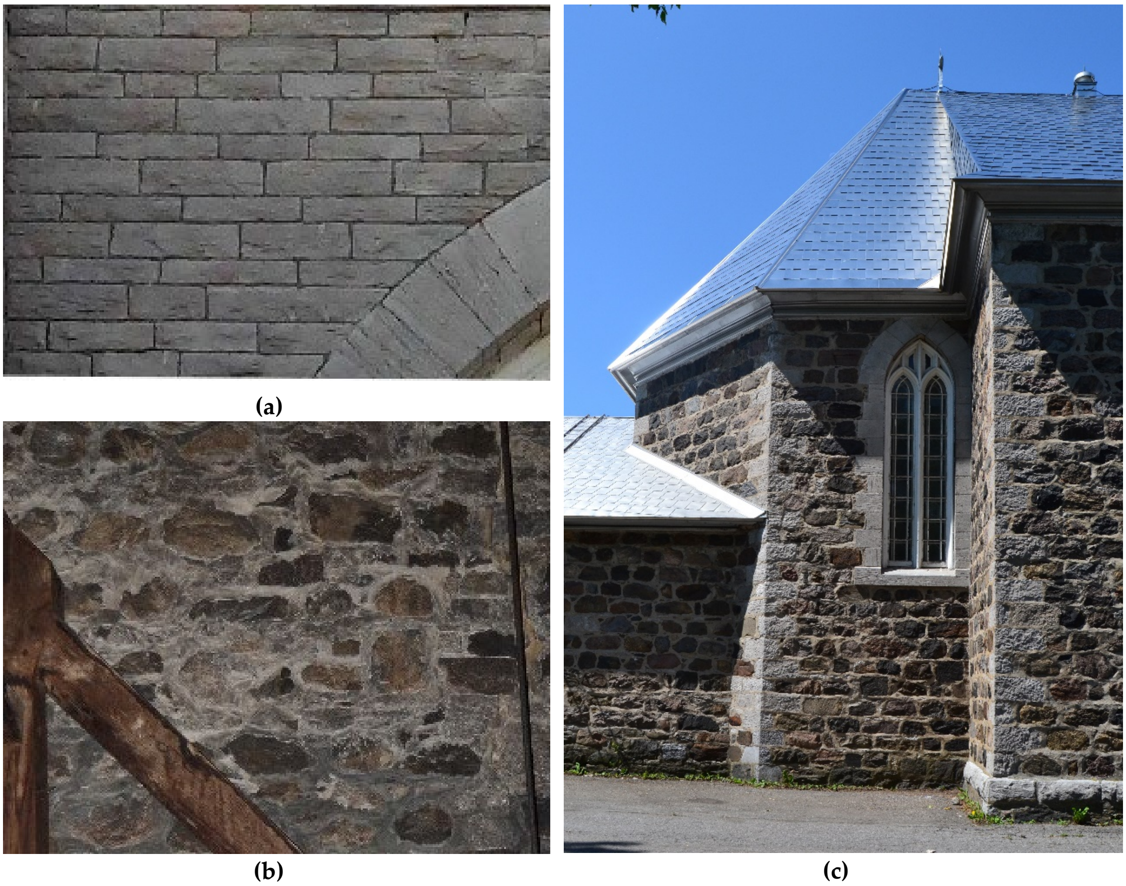


Figure 7- 31 Constructive details: **a.** External leaf of the façade; **b.** Internal leaf of the façade, visible from the roof level; **c.** Interlocking between the different walls of the church.

This aspect is a quite widespread construction practice in the visited churches, especially when the wall leaves are made of roughly squared stones. In St. Joseph, this attention is paid both between the façade and longitudinal walls of the nave (Figure 7- 32) and between the other walls of the church structure (Figure 7- 33). In the walls of the sacristy, the same stone is used for the wall elevation and the corner interlocking. This aspect is worth noting, considering that this material has not been the principal material of construction for this territory, especially before the major fires that occurred in 1682 and 1720, in Québec and Montréal respectively.

The rough stones, which constitute all the walls of the church, except for the façade, are laid in regular horizontal rows, but there is no regular vertical joint pattern. The lateral walls do not present any buttresses and have a repeated rhythm of big openings well marked with square stones, like those used for the interlocking between the walls (Figure 7- 34).

Figure 7- 32 Interlocking between the façade and lateral walls of the nave.

Figure 7- 33 Interlocking between the walls of the church structure.

Figure 7- 34 External view of the lateral walls of the nave: roughly squared stones. The openings are marked using square blocks of stones.

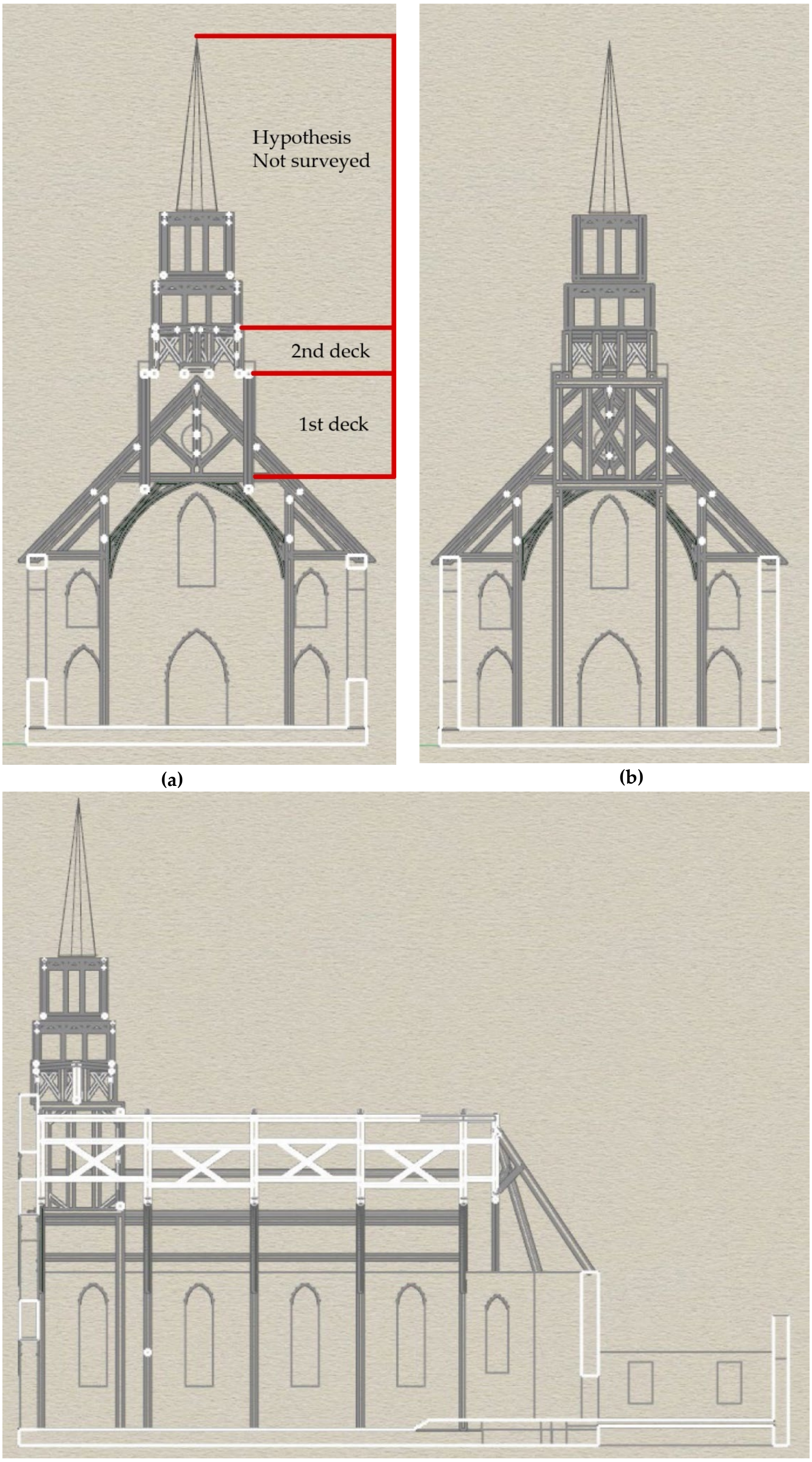
The interior of the church shows a three-nave organization of the space. Wood is the only visible material, used for the structure, decorations, and coverings. The nave is rhythmed by a colonnade that separates the principal nave from the lateral ones (Figure 7- 35a). The other two columns stand in the entrance area of the nave crossing the jubé level (Figure 7- 35b). A better understanding of the structure was possible visiting also the roof level, such survey allowed to implement a three-dimensional model (Section 7.4.2).

(a)

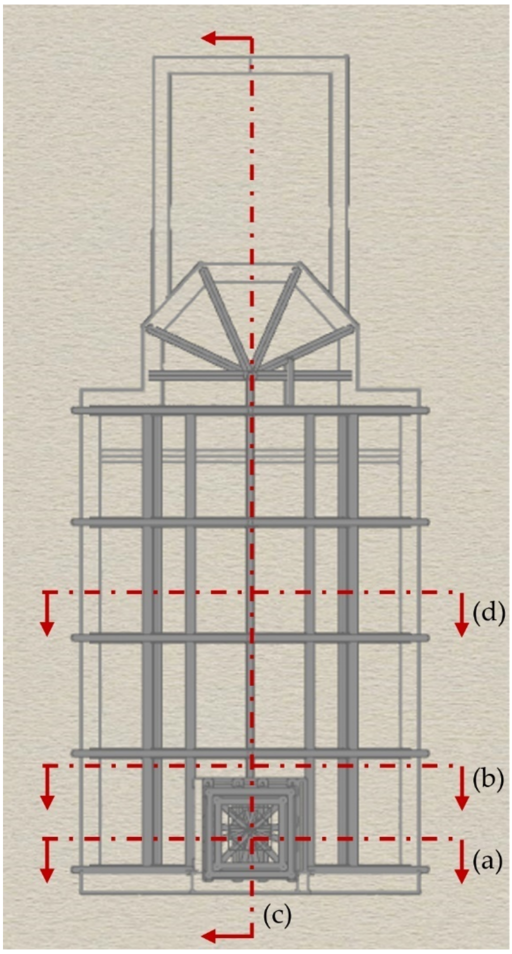
(b)

Figure 7- 35 Interior of St. Joseph church: **a.** view towards the apse; **b.** view towards the jubé.

In the following, some drawings of the church building, together with a 3D-model (Section 7.4.2), help to clarify its complexity (Figure 7- 36). Three transversal sections of the church are shown to explain the structure in different areas of the church: in the middle axis of the bell tower(Figure 7- 36a), immediately after the bell tower structure (Figure 7- 36b), and between one truss and the other cutting the longitudinal stiffening structure (Figure 7- 36d). A longitudinal section shows the relation between all the structural elements in the longitudinal direction of the church (Figure 7- 36c).



(a) (b)
(c)



(d) (e)

Figure 7- 36 Significant section from the 3d model: **a.** Section in the middle of the bell tower; **b.** Section immediately after the bell tower; **c.** Longitudinal section; **d.** Section between one structural unit and the other of the roof; **e.** reference plan where the section are highlighted in red.

The beams of the bell tower structure enter the façade wall both at the first and the second deck level (Figure 7- 36c). The bell tower stands supported, on one side, by two timber columns that reach the ground in the entrance area of the nave (Figure 7- 35b), and, on the other side, at the second deck of the bell tower, by three timber columns that enter into the façade (Figure 7- 37). The pillars of the bell tower visible in the nave in the jubé area continue at the roof level where they are clearly visible at the first deck of the bell tower, with their stiffening structure (Figure 7- 38, Figure 7- 39, Figure 7- 40).

Figure 7- 37 Pillars (built in the façade wall) and brace structure between pillars at the 2nd deck of the bell tower.

Figure 7- 38 Pillars and timber structure of the first deck of the bell tower (left).The red arrow shows the longitudinal brace structure that cross the brace structure of the bell tower.

Figure 7- 39 Pillars of the first deck of the bell tower and timber structure of the first deck of the bell tower (right).

Figure 7- 40 Columns of the bell tower and brace structure of the first deck of the bell tower and longitudinal stiffening of the roof structure.

Figure 7- 41 Last truss close to the façade wall and beams of the 2nd deck of the bell tower.

Figure 7- 42 Roof level above the central brace structure that longitudinally interconnects the trusses.

Figure 7- 43 Wider view of the roof structure: trusses and brace structure between trusses. In the background, it is visible the retrofitted truss through a metal stiffening element.

On the contrary, the roof structure does not interact with the façade. The top and bottom chord of the longitudinal stiffening structure end in the last transversal truss, which is separated from the façade wall by a gap of 50 *mm* (Figure 7- 41). On their side, the trusses are connected one to the other through a series of longitudinal braces (Figure 7- 42). Some of the trusses have the joints between the chord and the posts retrofitted with a metal element (Figure 7- 43). Towards the façade, the last brace of this longitudinal stiffening structure crosses the brace structure of the bell tower, without interconnecting with this last (Figure 7- 42, Figure 7- 40). On the contrary, towards the apse, the length of the last brace is smaller and the principal structure of the roof in this case has a radial set up (Figure 7- 44) Along the nave, the pillars of the colonnade are connected to the trusses that are the principal elements of the roof structure, creating a combined system that will be called here *roof system truss-columns* (Figure 7- 45, Figure 7- 46).

Figure 7- 44 Radial structure of the apse.

Figure 7- 45 Pillar from the nave, connected with the correspondent truss.

Figure 7- 46 Roof level above the lateral nave: longitudinal beams between the pillars of the colonnade. In the background the truss with the retrofitting metal element.

Considering the seismic vulnerability of the building, the presence of a high brick chimney in the backside of the church, in proximity of the apse, has to be pointed out (Figure 7- 30b, Figure 7- 30d, Figure 7- 47). As seen in Table 7- 2, where the reported damage of past earthquakes is listed, the chimneys frequently collapsed causing damage to the rest of the structure of the church. This functional element, added at a later time, is not a structural element but has to be considered as an element that increases the seismic vulnerability of the church.

(a) (b)

Figure 7- 47 The chimney in St. Joseph: **a.** Brick chimney well visible in the back of the church close to the apse; **b.** Brick chimney partially covers one window of the apse.

7.4.1 State of conservation and existing cracks

St. Joseph church is in a quite good state of conservation, even if some specific deficiencies were observed during the assessment phase. One phenomenon concerns some rising damp, visible at the base of the façade and buttresses (Figure 7- 48).

(a) **(b)**

Figure 7- 48 Rising damp visible at the base of the façade: **a.** on the center of the façade; **b.** on the right corner. The photo was taken on dry season (beginning of September).

The roof and the bell tower present signs of humidity, probably due to the high difference in temperature from the interior of the nave to the roof one. Indeed, in Canada for climate reasons the churches are kept warm with a heating system. For such reason, the difference in temperature is significant between the heated area of the nave and the unheated area of the roof. At the roof level (Figure 7- 49a) and at the second deck of the bell tower (Figure 7- 49b), signs of humidity are visible both on the wood planks and on the timber beam elements.

(a)

(b)

Figure 7- 49 Humidity on the timber planks and beams: **a.** Roof level; **b.** Second level of the bell tower.

Another aspect determinant for the state of conservation is the presence of cracks.

A vertical crack is easily noticeable in the left external wall close to the corner (Figure 7-50a). It was evidently repaired with a different type of mortar. A thin crack with a similar layout was observed parallel to the repaired one. This crack pattern could be the starting point for the development of an overturning mechanism of the façade, should an earthquake arrive with a longitudinal direction component. The origin of the crack could be, however, a different lateral load, for instance a strong wind, which could be sufficient to cause these cracks considering the slender bell tower and the size of the exposed surface. A strong wind condition caused severe damage up to collapse for various bell towers very similar to the one under study, in Eastern Europe (Moşoarcă et al. 2019). Here, the crack may be seen as an element of vulnerability.

On the right longitudinal wall of the nave, close to the façade, the redrafting of the stone masonry and a different mortar suggested the repair of a crack, probably symmetrical to the one previously described (Figure 7- 50b).

This last hypothesis is based on the internal observation of a symmetrical pattern of cracks, both in the façade wall and in the longitudinal walls of the nave, close to the windows (Figure 7- 51).

A vertical crack was also observed in the façade wall at the location of the interconnection of one timber beam to the bell tower (Figure 7- 52). It seems not to be a recent one.

(a) (b)

Figure 7- 50 Crack on the lateral walls of the nave: **a.** A vertical crack close to the corner was repaired and a new similar one appeared almost parallel to the previous one; **b.** A modification of the stone fabric is visible in the area close to the corner in the opposite lateral wall of the nave. A symmetrical structural problem can be assumed.

(a) (b) (c)

Figure 7- 51 Cracks near the openings in the lateral walls close to the façade: **a.** first window in the north lateral wall; **b.** first window in the south wall; **c.** second window in the south wall.

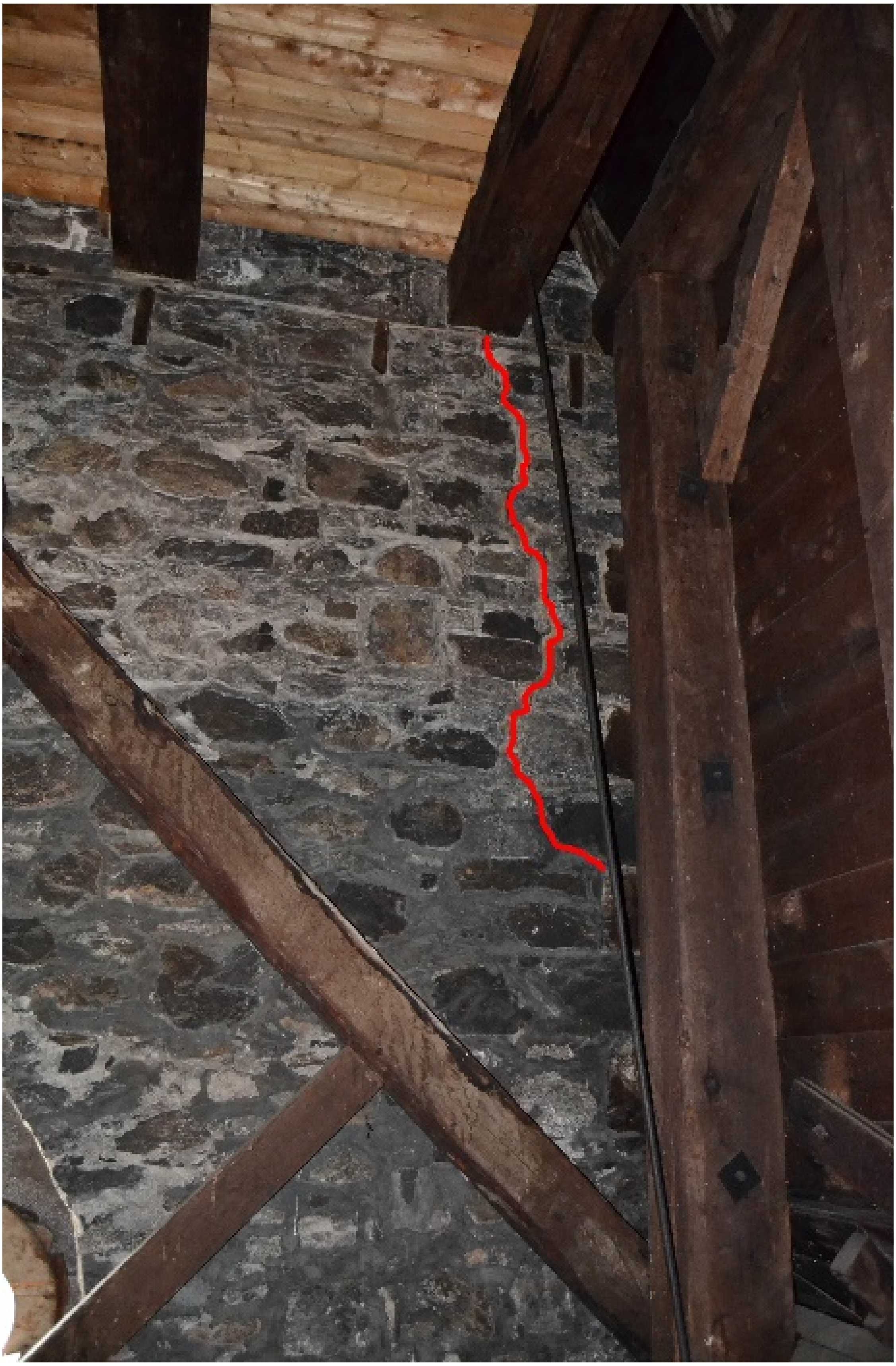


Figure 7- 52 Crack in the interior side of the façade. It has a vertical path starting at the base of one beam of the second deck of the bell tower.

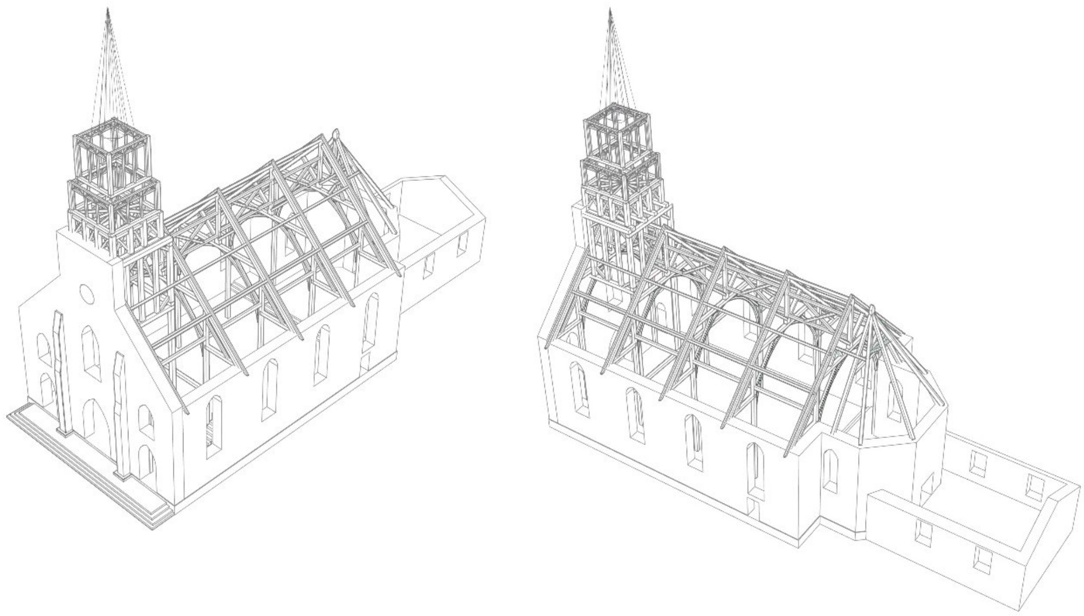
7.4.2 THE 3D MODEL

The implementation of a 3D model was an essential step for the structural understanding of St. Joseph, due to the complexity of the roof structure, the bell tower, and the interrelation with the masonry part. All the structural elements were surveyed, including those of the roof and bell tower, and the scrupulous observation of the connections (Figure 7- 53), especially at the roof level, provided an accurate base for the decisions taken during the implementation of the numerical model.

(a) **(b)** **(c)** **(d)**

Figure 7- 53 Details of timber connections of the structure of the roof and bell tower: **a.** Tenon – mortise connection with timber connector between rafter and the strut of the truss; **b.** Tenon-mortise connection between the longitudinal stiffening structure and the column of the colonnade from the nave; **c.** beam of the first deck of the bell tower entering the façade and pillars of the bell tower; **d.** Detail and accuracy of the tenon – mortise connection between the beam of connection of the pillars of the second deck of the bell tower and the stiffening structure.

The model was stripped of decoration details. The model presents the real size of the columns of the colonnade and the beams without the wood plank covering used for the interior of the church. The elaborated 3-D model was the base for the numerical one. Figure 7- 54 shows two views of the entire model.



(a) (b)

Figure 7- 54 3-D model of St. Joseph church: **a.** North west view; **b.** South east view.

The elaboration of this model was determinant for the understanding of such a mixed structure. The construction material for the walls is the masonry while for the rest of the structure is timber. This last material is a determinant part of the complete church structure because it includes the columns, the roof and the bell tower.

7.4.3 The Structural Analysis

The combined use of the two materials (masonry and timber) of St. Joseph motivated studying the structure by means of a structural software system to explain, on the one hand, the global behavior of the building structure, and on the other hand, the influence of the roof and bell tower. For such aims the following steps were performed:

- Modeling the masonry walls of the church, including the mass of the roof and the bell tower at different levels of the façade;
- Analyzing the behavior of the façade and the bell tower;
- Understanding the stiffness and the contribution that the roof could provide in containing the bending out-of-plane of the lateral walls and possibly their partial overturning.

The first step consisted of modeling the church on a global scale, elaborating a finite element model in a structural analysis system (Abaqus 2016). The mesh of the masonry parts consisted of 36585 tetrahedral elements, with two elements in the thickness of the walls, through a linear formulation (Figure 7- 55).

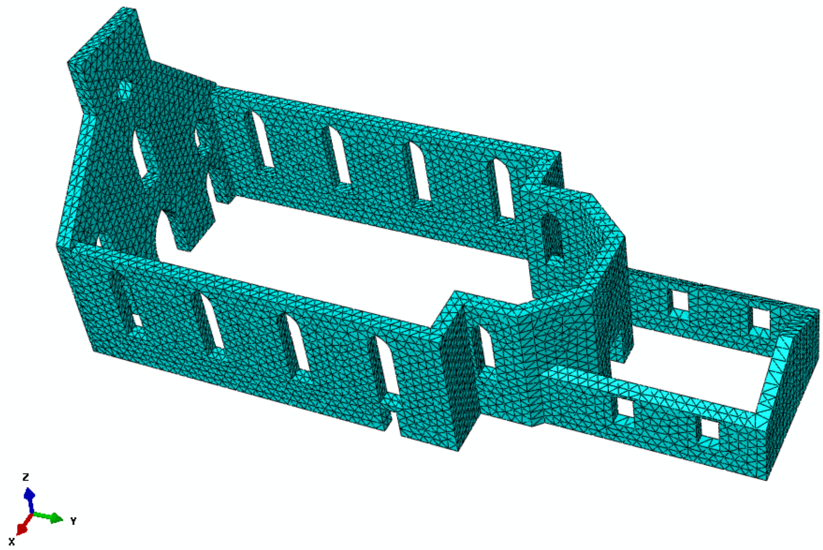
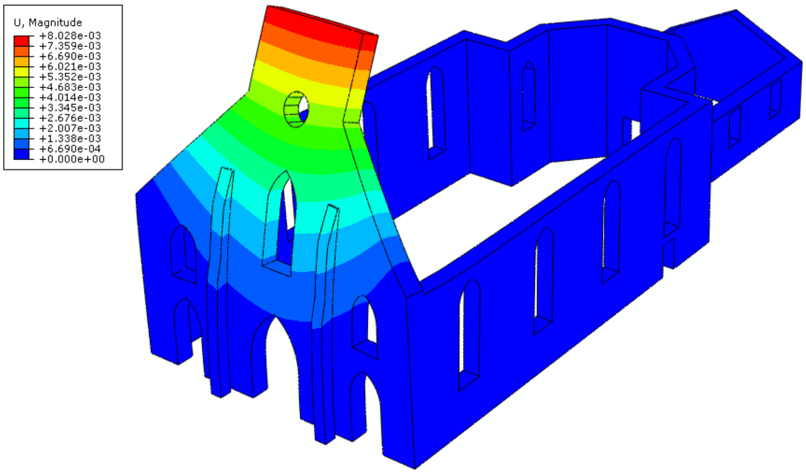


Figure 7- 55 The mesh of St. Joseph used for the modal analyses.

For the global analyses of the model, the heterogeneity of the masonry and its orthotropic behavior were neglected. The properties of an equivalent homogeneous isotropic material have been defined according to the strength and the Young's modulus values reported in the Guidelines for the seismic assessment of stone-masonry structures (Chidiac and Foo 2002), that was produced for the Federal Government for the maintenance of their masonry buildings. The following properties of the masonry were considered: Young's modulus E= 2200 MPa, and the density w = 20 kN/m^3, and the mass contribution of the roof and the bell tower were included in the model considering the density of the wood structure w = 4.5 kN/m^3. Figure 7- 56 and

Table 7- 3 show the first most significant modal shapes, the natural periods, and the participating mass ratios for the longitudinal and transversal directions of St Joseph.


U, Magnitude
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+6.690e-03
+6.021e-03
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+4.683e-03
+4.014e-03
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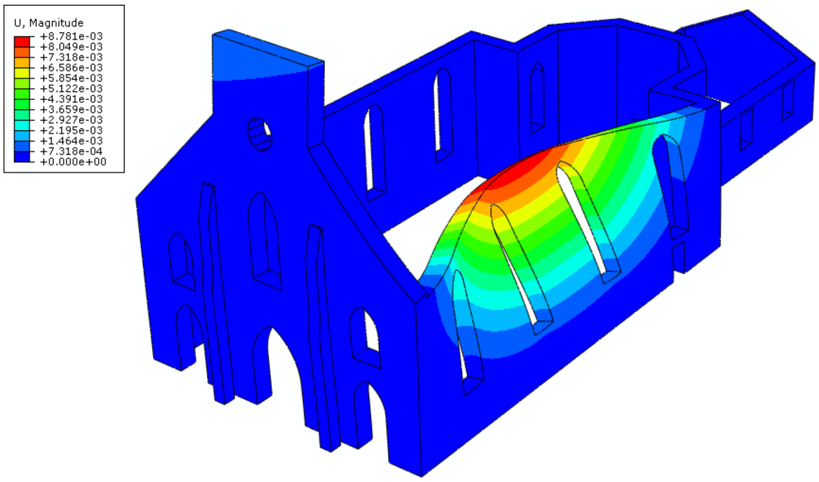
+2.676e-03
+2.007e-03
+1.338e-03
+6.690e-04
+0.000e+00

(a)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+4.391e-03
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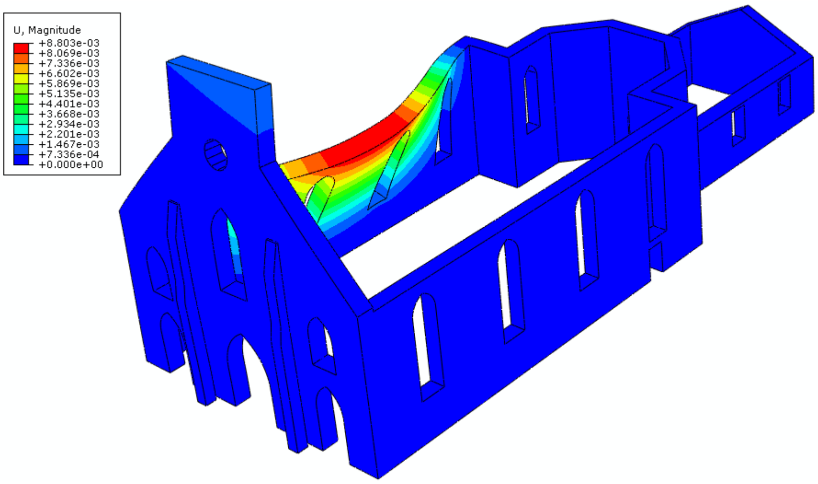
+1.464e-03
+7.318e-04
+0.000e+00

(b)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+8.069e-03
+7.336e-03
+6.602e-03
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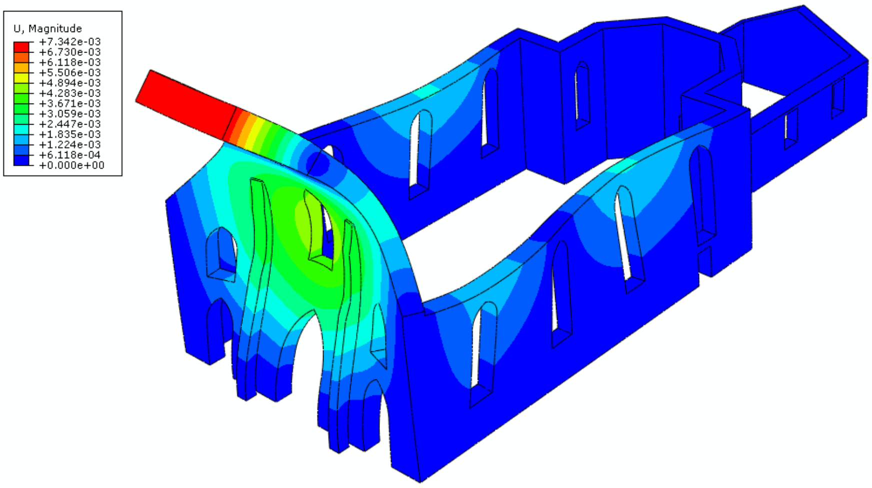
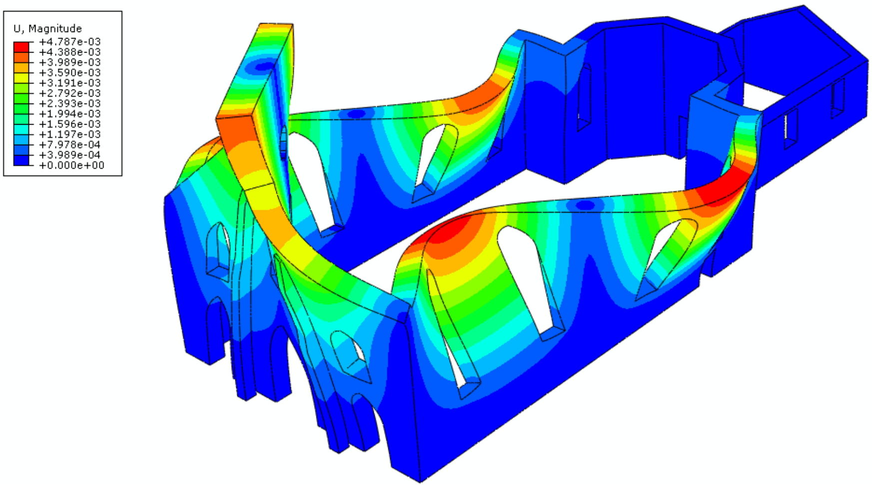
+5.135e-03
+4.401e-03
+3.668e-03
+2.934e-03
+2.201e-03
+1.467e-03
+7.336e-04
+0.000e+00

(c)



(d)



(e)

Figure 7- 56 Modal shapes of St. Joseph model (including the mass of the roof and the bell tower: a. Mode 1; b. Mode 2; c. Mode 3; d. Mode 4; e. Mode 5.

Table 7- 3 Periods, participating mass ratio in the two major direction of the church structure.

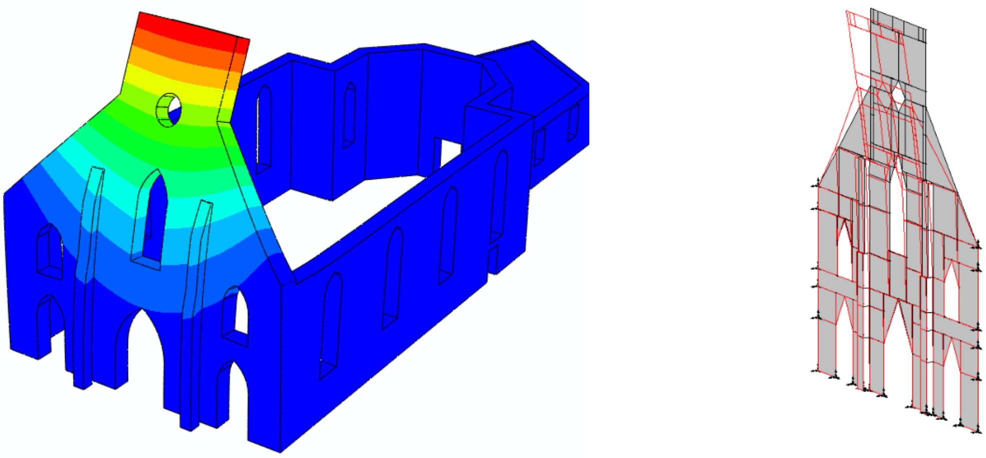
Figure of the modal shape	**Mode**	**Period (s)**	**Participating mass ratio, longitudinal direction (%)**	**Participating mass ratio, transversal direction (%)**
57a	1	0.74	4,16	0,00
57b	2	0.21	0,01	3,72
57c	3	0.21	0,01	3,91
57d	4	0.15	2,50	0,00
57e	5	0.13	0,00	0,01

The first and the fourth modes involve the façade with the participating mass concentrated along the longitudinal direction (Figure 7- 56a,Figure 7- 56d), while the second and third interest the lateral walls of the nave (Figure 7- 56b,Figure 7- 56c). Mode 1 with a period of 0.74 s engages the upper part of the façade. Mode 4 has a period of

0.15 s and it shows a bending mode, that involves mainly the upper part and center of the façade. On the contrary, Mode 2 and 3 concern only the lateral walls with a period of 0.21 s showing an out-of-plane mode. Finally, the fifth mode is a torsional mode that involves both the façade and the lateral walls (Figure 7- 56 e).

From these modal results, it emerged that understanding the structural contribution provided by the bell tower and the roof was an essential step to be performed. Consequently, the structural behavior of the façade and the bell tower were studied using a local model. Similarly, the contribution of the roof to the church building in answering to horizontal forces was investigated.

The local finite element model of the façade was developed in the structural analysis system ADA (Figure 7- 57b). To insure the reliability of the results, this local model was calibrated by comparing its first mode (overturning of the façade) with the first mode of the global FE-model (Figure 7- 57). The two values in terms of periods correspond to 0.79 s and to 0.74 s, respectively.

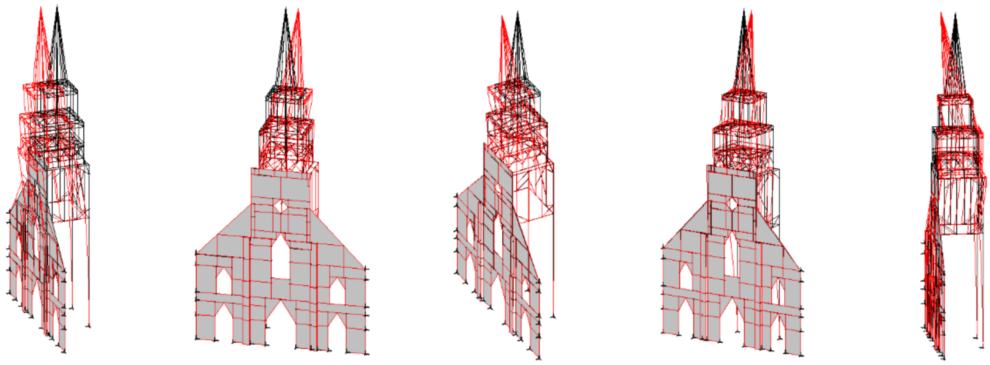


Mode 1 T=0.74 s

Mode 1 T= 0.79 s

Figure 7- 57 Comparison between Mode 1 from the global FE-model (Abaqus 2016) and Mode 1 of the façade elaborated in ADA (Ben-Ari 1998).

The following step was to add the bell tower structure to the local model of the façade in masonry. A total of 490 beam elements, mostly with pin connections to model the low moment transmission capability of carpentry joints, and 385 nodes were necessary to model the bell tower. Based on visual inspection, the timber material was identified as softwood of the Pinaceae family, most used in the area, for which an elastic modulus parallel to the fiber equal to 9000 MPa was considered. Figure 7- 58 shows the behavior of the bell tower and the most significant modal shapes for the local model.



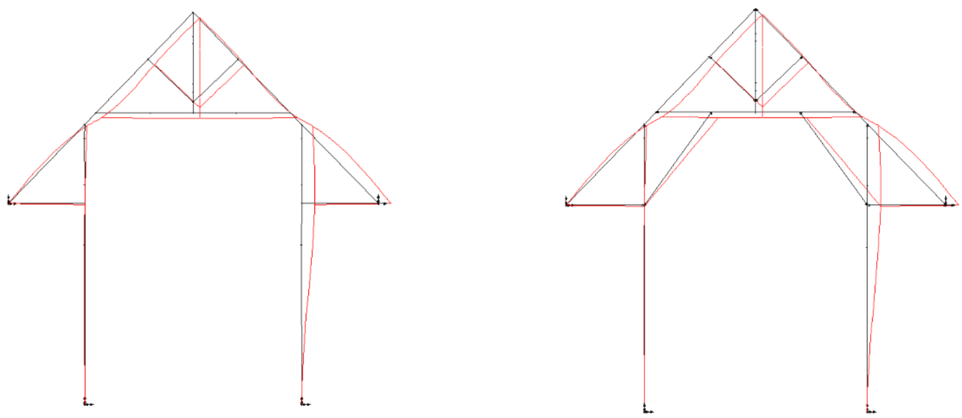
T=0.72 s **(a)** T=0.38 s **(b)** T=0.37 s **(c)** T=0.27 s **(d)** T=0.26 s **(e)**

Figure 7- 58 Modal shapes of the local model: **a.** Mode 1; **b.** Mode 2; **c.** Mode 3; **d.** Mode 4; **e.** Mode 5.

Mode 1 (Figure 7- 58a) and mode 3 (Figure 7- 58c) of the local model show the out-of-plane of the façade, with a period of 0.72 s and 0.37 s, respectively. Mode 2 is parallel to the façade with a period of 0.38 s (Figure 7- 58b), Mode 4 is a torsional mode (Figure 7- 58d), while mode 5 is flexural with a period of 0.26 s (Figure 7- 58e).

The behavior observed in the local model of the façade and bell tower is comparable to that observed in the global one. The period of the first mode decreases from 0.74 s (Figure 7- 56a), in the global model, to 0.72 s (Figure 7- 58a), in the local one, showing in both cases an overturning of the façade. This difference may be associated with the presence of the bell tower in the local model. The second mode shows a displacement parallel to the façade, in the local model developing in the top part of the bell tower (Figure 7- 56b), while, in the global one, it principally involves the lateral walls of the nave (Figure 7- 58b). There is less correspondence between models starting from the third mode, where, in the global model the displacement concerns the lateral walls of the nave (Figure 7- 56c), while the local model shows an overturning of the façade and bell tower (Figure 7- 58c). Finally, the fourth and the fifth modes in the two models differ: in the global model, mode 4 shows an inflection (Figure 7- 56d) while Mode 5 a torsion (Figure 7- 56e), in the local model, the contrary occurs (Figure 7- 58d,Figure 7- 58e).

In order to understand the influence of the roof structure on the response of the masonry walls to horizontal forces, a bi-dimensional model of the single roof structure truss-columns was elaborated. It is composed by the truss and a pair of timber pillars. From the onsite survey, it was not clear whether the arch, which had definitively an esthetic function in the church interior, had also a structural function. For this reason, the two configurations were compared applying a horizontal force of 10 kN at the right side of the truss in the point of connection with the wall of the nave. Figure 7- 59a shows the configuration without the arch, while Figure 7- 59b shows the model with the arch. The displacement is respectively of 2.16 mm and 2.13 mm, indicating a very limited contribution of the arch elements.



d= 2.16 mm (a) *d*= 2.13 mm (b)

Figure 7- 59 Deformed shapes of the structural unit considered for obtaining the stiffness of the structural system, applying a horizontal force at the connection of the structure with the wall of the nave.

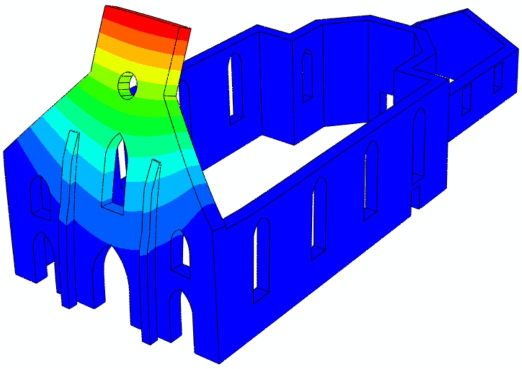
Consequently, from such results the horizontal stiffness of the roof structure at the level of the masonry wall was calculated according to the following expression:

$$k = \frac{F}{d} = \frac{10\text{kN}}{0.00216\text{m}} = 4630\ \text{kN/m}$$

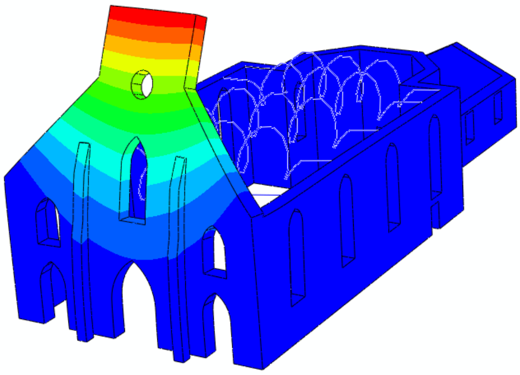
This horizontal stiffness was used for setting the stiffness property of springs inserted in the global FE model at the position of the trusses at the top of the masonry walls in the church nave to simulate the roof contribution to the transversal stiffness of the walls.

Figure 7- 60 shows the comparison of the modal analysis from the FE model, considering the contribution just in terms of mass provided by the roof and bell tower structures (a), and the modal shapes in the model that includes the springs (b).

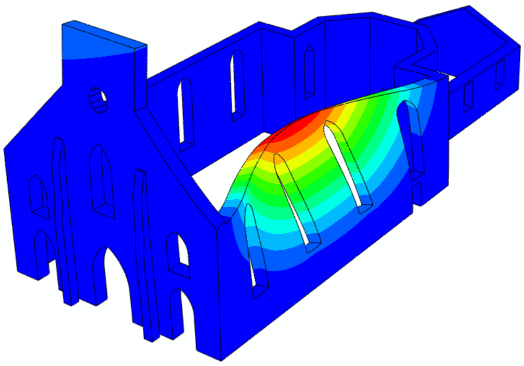
In both cases, the first mode develops in the façade. It is from the second mode that the modal shapes slightly differ, influencing the participating mass ratio in the transversal direction. The positive contribution of the roof is evident in particular in the second and fourth mode, where the two walls of the nave respond together to actions rather than in opposition.



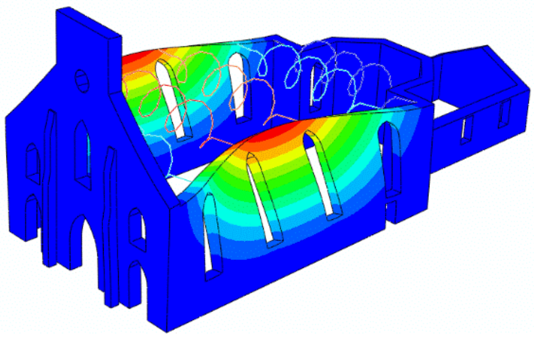
Mode 1
T=0.47 s
% long. partic. mass ratio=4.16
% transv. partic. mass ratio=.0.00



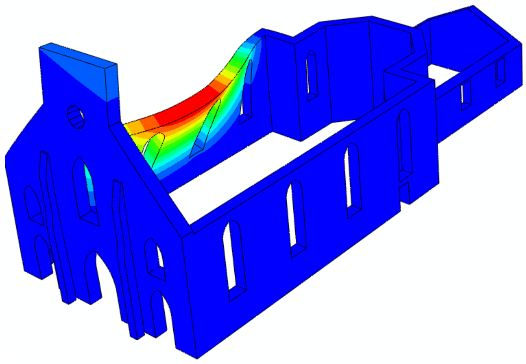
Mode 1
T= 0.47 s
% long. partic. mass ratio=4.12
% transv. partic. mass ratio= 0.00



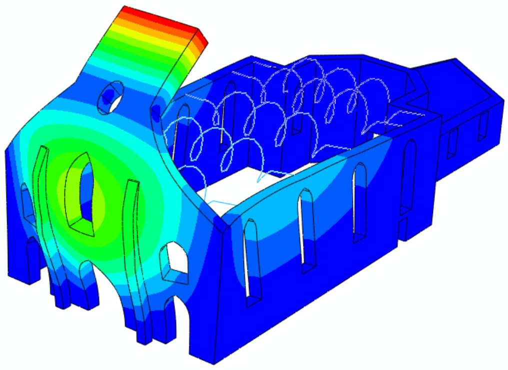
Mode 2
T=0.21 s
% long. partic. mass ratio= 0.01
% transv. partic. mass ratio=3.72



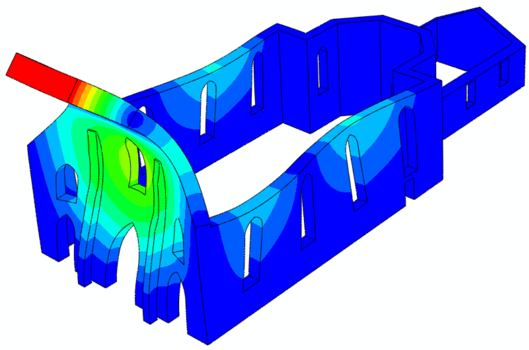
Mode 2
T=0.21 s
% long. partic. mass ratio= 0.00
% transv. partic. mass ratio=7.63



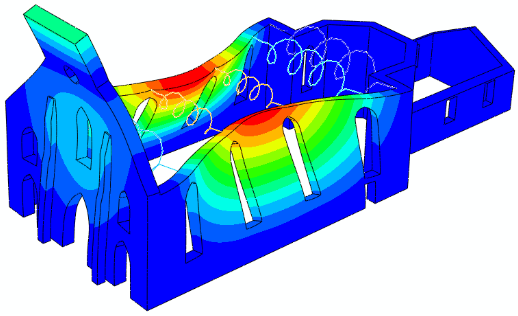
Mode 3
T=0.21 s
% long. partic. mass ratio= 0.01
% transv. partic. mass ratio=3.91



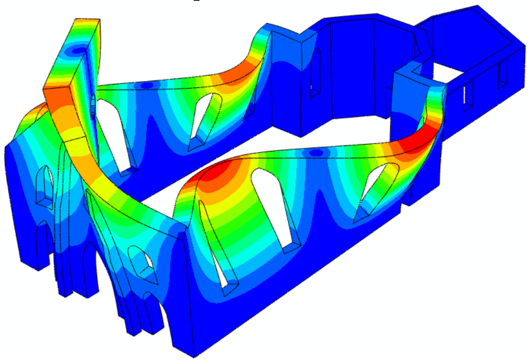
Mode 3
T=0.15 s
% long. partic. mass ratio= 2.55
% transv. partic. mass ratio=0.00



Mode 4
T=0.15 s
% long. partic. mass ratio= 2.50
% transv. partic. mass ratio=0.00

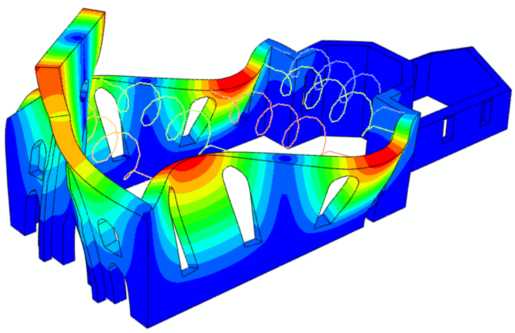


Mode 4
T=0.14 s
% long. partic. mass ratio= 2.24
% transv. partic. mass ratio=0.00



Mode 5
T=0.13 s
% long. partic. mass ratio= 0.00
% transv. partic. mass ratio=0.01

(a)



Mode 5
T=0.13 s
% long. partic. mass ratio= 0.00
% transv. partic. mass ratio=0.01

(b)

Figure 7- 60 Comparison between the two models: **a.** including the mass of the roof and bell tower; **b.** adding the springs that simulate the roof contribution.

The analyses have clarified the role of the roof structure in the global seismic response. The effect is highly dependent on the type of roof structure and on the quality of connection with the nave walls. The quality of execution of timberwork in the Québec churches appears very high and allows to assume effective connections even where these are not inspectable. The type of trusses present in St Joseph offers sufficient lateral stiffness to the nave walls in the transversal direction to act as a link limiting their out-of-plane behavior. A simple bi-dimensional scheme may be used to evaluate such stiffness, as done above. Other truss configurations observed in different churches of the set examined seem to offer either a lower or a higher contribution. Analyses are currently in progress in another research project.

7.5 The TSK-Form applied to the Canadian churches

The previous sections have shown how the Canadian churches in the region considered differ from the Italian cases. They are the result of a different construction culture; therefore, the understanding of their specificities becomes a crucial step for an appropriate seismic vulnerability assessment. In such a context, it was decided to use the TSK-Form that should provide the necessary flexibility and adaptability. In particular, the TSK-Form was applied to the church of St. Joseph because of the availability of results from the more extensive study, including onsite surveys and elaboration of models. As mentioned in section 7.3, the néo-roman typology would be difficult to interpret by means of the abacus of 28 mechanisms. The aim was, once more, twofold: to assess the seismic vulnerability of the case under examination and at the same time to sort out the territorial specificities that require attention for such typology.

As exposed in section 6.3, also in this case the elaboration of the TSK-Form, that is the practical realization of the territorial knowledge approach, passes through two principal steps, that are:

- *Step 1.* Adaptation of the TSK-Form to the Canadian context and the néo-roman typology.

  The previous sections have facilitated the understanding of the construction culture characterizing this territory and the understanding of those aspects that must be included in the form and that differ from the Italian case. This action is intended here as an illustration of the procedure, without pretending to be exhaustive. It should require a deeper refinement that could be easily obtained through the analysis of a larger number of church case studies. For the elaboration of the TSK-Form, the general layout was kept similar to the one presented for the Italian case, with a general section and a detail one. In this context, what emerges immediately is the difference in the identified territorial macro-elements and consequently the associated territorial vulnerability modifiers. For the néo-roman typology, the following territorial macro-elements were identified:

- *Façade-bell tower.* In this typology the distinction between the masonry façade and the timber structure could not be distinguished due to the strict interrelation in structural response of these two and being the most vulnerable part of the entire structure. For such reasons the introduction of a macro-element specific to this typology was necessary;
- *Other walls.* This macro-element includes the longitudinal walls and the apse ones whose vulnerability is mainly dependent from the roof capacity in collaborating to answer to horizontal seismic actions;
- *Roof system truss-columns*. This macro-element is characteristic of this typology, constituted of three naves where the columns are made of timber and are connected to the trusses at the roof level. In section 7.4.3, such structural system was studied leading to the conclusion that the structural behavior of trusses and columns cannot be studied independently and that together they have a strong influence in the horizontal structural response of the church;
- *Annexed body: the sacristy walls.* The sacristy was frequently found as annexed body in the back side of the apse, even if it was not a later addition of the church. Consider a specificity of the typology was included here;
- *Annexed body: the chimney* It was added in consequence of the analysis of the historical damage for churches and the observation of such element as recurrent in this territory.

Finally, the last column of the TSK-Form is devoted to possible mechanisms for every identified macro-element, considering the difference with the Italian cases that does not allow the mechanism interpretation, recurring to the list of the 28 reference cases.

- *Step 2.* Assessing the seismic vulnerability of the analyzed church.

  The case of St. Joseph was assessed recurring to the TSK-Form adapted to the néo-roman typology, explained in section 7. 5. 1.

7.5.1 THE APPLICATION OF THE TSK-FORM TO ST. JOSEPH

The TSK-Form of St. Joseph ca be consulted in Appendix V. The general section is thought as it would be pre-filled by the local offices that operate in the territory for the seismic vulnerability assessment of this typology. It reports the low level of seismic awareness that, in the building stock, is expressed by a not-significant presence of seismic provision details. In the same general section, the most recent seismic event that was felt in Montreal is reported, the Saguenay earthquake (1988).

The detail section focuses on the information of St. Joseph church. collecting significant specificities for its typology also due to the accurate survey and the elaboration of a 3D model (section 7.4.2) performed. Some considerations from the TSK-Form are summarized in the following:

- The most vulnerable territorial macro-element is the 'façade-bell tower' with a medium high level of vulnerability derived from the assessed territorial modifiers.
- The presence of many openings, in particular of the big one in the middle of the façade, makes the façade wall highly subjected to shear cracks, and for its position respect to the bell tower such condition could facilitate the development of a hinge for out-of-plane mode of the gable. This damage is also favored by the existing cracks in correspondence of the two symmetrical openings at the second order. Moreover, the presence of a vertical crack in the north-east wall, that was repaired and has appeared again parallel to the previous one could be the starting point of an out- of-plane mechanism of the façade.
- The presence of a pair of buttresses positively contributes to contain the out-of-plane mode of the façade.
- The well-marked corner-connection between the different perimetral walls is a positive aspect of the construction practice that contributes to a good

performance of the church against earthquakes. The weakest part results the soaring portion of the façade - assessed with a 'high level of vulnerability - that is not anchored nor retained by seismic provision details and its condition is worsened by the beams and pillars of the bell tower that could also cause local damage.

- The connections of the roof system truss-bell tower and the longitudinal braces between units are well built and in good state of conservation, except for two retrofitted trusses. They play a significant role in containing the out-of-plane of the upper portion of the longitudinal walls and in distributing among the walls of the nave and the apse the response to horizontal actions. This property included in the form as 'boundary condition' in the 'other walls' macro-element reduces their vulnerability. Referring to the configuration and the modifiers the macro-element 'roof system truss-columns' was assessed with a medium-low level of vulnerability. The attention in this case should be payed to the connections.
- The presence of the beam of the jubé entering the longitudinal walls is another boundary condition that could determine a local damage.
- The 'chimney' macro-element is assessed with a medium level of vulnerability due to its slenderness and referring to the historical damage.

7.6 Considerations on the seismic vulnerability for the Canadian churches

What emerged after the detailed analysis presented in this chapter is a different construction culture that characterizes the features of the churches in this geographical area, and consequently, their seismic vulnerability. At this point of the research, some considerations on the seismic vulnerability of churches from Québec can be expressed.

The seismic awareness seems low, observing the low presence of seismic provision devices in the historical building stock and on churches, only 1 over 14 visited churches have tie rods in the transversal direction. It is the church of Notre Dame du Bon Secours that belongs to the Conefroy typology and that, by its status as cultural heritage, has undergone some partial reinforcement interventions.

Two construction materials are used in the construction of the churches: masonry and timber. The former is relegated to the walls, the latter to the columns, vaults, bell tower and roof structures. The interlocking between walls is usually guaranteed using square-cut stones and the role of the horizontal diaphragm between the longitudinal walls of the nave is taken by the roof structure.

For the néo-roman typology, the colonnade is connected, at the top with the roof structure, and laterally, to the walls of the nave, contributing in such a way to guarantee the response of the churches to horizontal forces in the transversal direction.

The roof structure is usually not connected to the façade. It connects the longitudinal walls of the nave and is made of a series of trusses tied one with the other through a longitudinal system that ends in the apse with a radial configuration. This layout avoids the pounding damage caused generally by the beams of the roof, but at the same time, it leaves the upper portion of the façade free to move. In the néo-roman typology, the upper portion of the façade is highly vulnerable because it is not anchored and suffers the influence of the bell tower.

The bell tower differs from one typology to another, but for its features it is a characteristic element of the churches in this territory, in particular of their façades. The seismic vulnerability of such element that involves the façade is strictly dependent on the way it is built and more in general on the church typology. An example can be the comparison between the Conefroy and the néo-roman typology. In the first, the bell tower is completely independent from the façade, in the latter it may change from one church to another. In the analyzed case study, St. Joseph, the bell tower interacts with the façade at different levels: at the top, its pillars stem from the façade and, at the first bell tower deck, the longitudinal beams of the bell tower itself enter the façade. From analysis of the numerical model, it was observed that the collaboration of the façade and bell tower increases the period of the first mode (from 0.74s to 0.79s). The roof structure, in this case study, provides a positive contribution to the response of the lateral walls in the transversal direction, linking them and preventing them from moving apart in an out-of-plane mode, making their behaviour more uniform.

Finally, the TSK-Form adapted to the néo-roman typology clarifies the territorial approach proposed in this research, differing from the Italian one both for the macro-elements and its vulnerability modifiers.

CHAPTER

8

THE COLLECTION OF INFORMATION IN A GIS WORKSPACE

This section provides a few notes on the use of the GIS software during the research and at the end for the collection of territorial specificities of theinvestigated churches.

In this research, a GIS (Geographic Information System) was an essential tool both during its development and at the end to integrate the different levels of information. It was chosen because of its flexibility and adaptability that fits the necessity to accommodate different conditions and territorial contexts, a characteristic of this research.

During the development of the thesis, the GIS was used to localize the churches in the territories and overlay a series of information useful for further interpretations on what had occurred. For example, in the cases of Central Italy and Lombardy, the PGA isocurves of the earthquakes considered were overlaid with the location of the churches to be analyzed. Such action contributed to understand the damage and to take decisions on the procedure to be applied. In the case of the church of San Salvatore, the use of GIS allowed to intersect the information on the location of the recording station of Colfiorito and the PGA isocurve of the church. This comparison justified the decision to take the accelerograms recorded in that station for the analysis of the church.

At the end of the research, the GIS software allows to connect the TSK-Form of the churches with the maps elaborated in the process. Figure 8- 1 shows an example of collected information for the church of San Bartolomeo in Quistello.

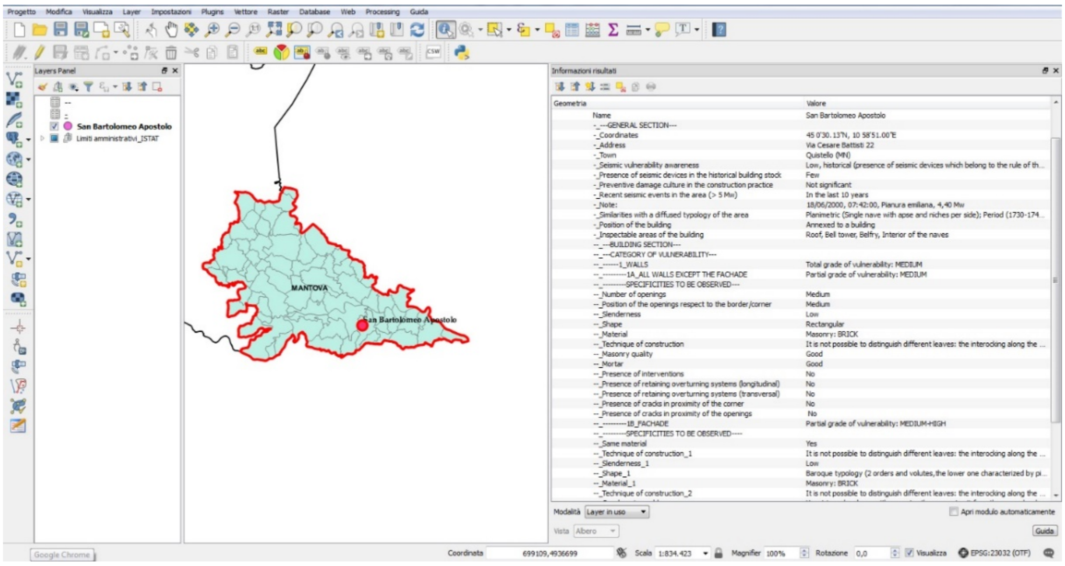


Figure 8- 1 Example of information on the church of San Bartolomeo, collected through the TSK-Form and linked in a GIS layout.

As a conclusion, using this tool contributes to the accessibility of the "new knowledge" on territorial specificities related to the seismic vulnerability of churches. The collected information, systematized in a GIS layout, could easily dialog with databases of

Authorities concerned, such as the Civil Protection Agency and the Ministry of cultural heritage through the Superintendences operating in the different territories.

CONCLUSIONS

Historical masonry churches are a major element of the Italian Cultural Heritage. Earthquake events that regularly occurred on the Italian territory have been systematically eroding these assets, which are particularly vulnerable due to their construction characteristics.

An impressive choral effort has been offered by the Italian scientific community along several years to provide solutions for the main relevant issues, that is, criteria for damage assessment, for effective and heritage-compatible interventions, and in a prevention perspective, for seismic vulnerability assessment. Among the many tools that have become available, the search and the definition of ad-hoc damage mechanisms have been an important step forward for dealing with these problems. The mechanisms identified as proper to describe damage of this asset class have allowed a systematization of the key issues. The precision and the expressivity of the damage patterns have actually created a very effective common language among scholars and practitioners operating in the field.

Starting from this consolidated base, and in order to improve the treatment of this very delicate subject it is now worth considering some issues that have appeared as needing refinement at the current state of knowledge and practice.

In a damage prevention perspective, this work has offered a contribution to highlight some inconveniences that could emerge in vulnerability assessment operations and to offer criteria for their solution. The main issue is in the need for more inclusive criteria that may allow classification of situations different from the currently defined set of possible macro-element typologies and related damage patterns. In spite of the tremendous amount of observed damage cases on which it is based, the current abacus of mechanisms derives from observations of Italian earthquakes. Its imprint derives from the constructional culture of a well-defined, albeit very extended, territory. The effect is twofold: a difficulty to apply the current methodology in territories characterized by different construction traditions and practices, and, within the original territory, to classify some cases related to typologies that may be less frequent and consequently not reflected in the reference situations or easily transferable to them.

A solution that seems possible to cover these inefficiencies is to adopt the analyses of the local construction specificities, with their different meanings, as the first step in a vulnerability assessment. The thesis has followed this approach, proposing a medium-level procedure, that is, making reference to assessment operations performed by trained personnel, not in an emergency time, thus not needing a rapid exploitation. The method identifies the macro-elements and the vulnerability modifiers for that territory. With this information, the assessment, guided by a suitable survey form, TSK-Form, includes but is not limited to the recognition of the basic mechanisms, but enters in detail in those aspects that are typical of the area and that have an influence in the structural response. In order to focus on the concept of territory-related characteristics that have been observed or may be expected to trigger seismic damage, a series of case studies have been analyzed.

From a group of churches in Lombardy, damaged by the Pianura Padana Emiliana earthquake of 2012, notes on the response of locally widespread baroque churches have been developed.

An interesting case from Central Italy was a church that had a history of interventions and damage from different earthquakes. The analysis of its response with the relevant ground motions and the damage both computed and observed allowed to estimate the effect of the different types of interventions performed, which are also associated, for good or bad, to the vulnerability of the asset.

A last Italian case is constituted by a church-fortress in Eastern Sicily, that offered occasion not only to study the structural behavior of this configuration, but also the effect of strengthening interventions performed according to a modality now superseded.

For all these cases, the proposed investigation approach has been applied and has shown a good capability of description.

Finally, the method has been tested in Québec, a territory that is very far for history and traditions. The numerous churches of the island of Montreal have been considered, initially assessing their seismic vulnerability according to the current Italian methodology, but incurring in some difficulties of interpretations. Numerous surveys and documental sources have then allowed to formalize a typological classification of this church group. For one such class, the néo-roman, numerical analyses have brought to recognize and to quantify the effects of its main stylistic characteristics on the seismic response. Results have been used to adapt the TSK-Form to the néo-roman typology, whose macro-elements and vulnerability modifiers exalt the territorial approach proposed by this study. Other typologies of the churches of Québec are now locally under study by a research group at ETS, and this is a first outcome of this research. The approach that has been proposed will need improvement that may stem from calibration with a large number of cases and from consideration from this first experience. Reference to specific construction characteristics appears to the author as a capable means to respond to needs for a more inclusive and adaptable vulnerability assessment.

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APPENDIX

IA

MAP OF ITALY FOR THE INVESTIGATED AREAS

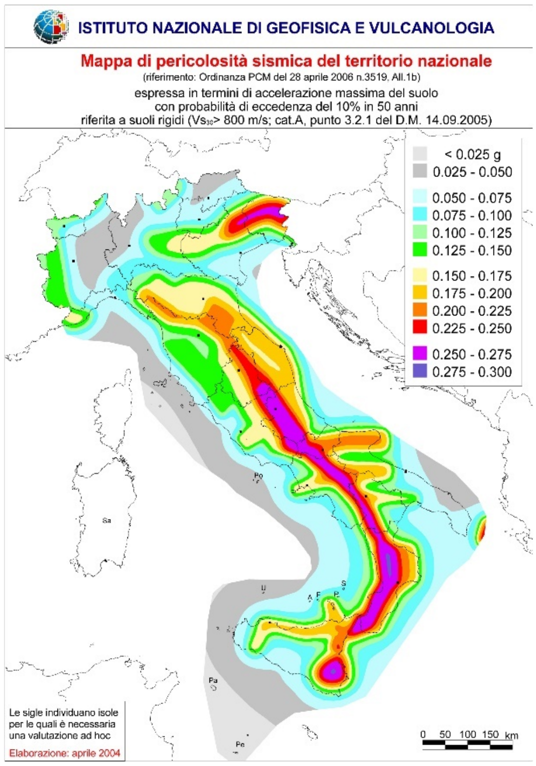

Figure On the left, in red three geographical areas where the investigated churches are. From North to South: Lombardy, Marche, and Sicily. On the right, the Italian Seismic Hazard map for a probability of exceedance of 10% in 50 years.

APPENDIX IB

INFORMATION ON THE VISITED CHURCHES IN CENTRAL ITALY

Identification the church				Materials & Structure				Information on earthquake damage and interventions					
Municipality	**Address**	**Name of the church**	**Year**	**Plan Configuration**	**Other relevant info (i.e. facade-bell tower)**	**Construction material**	**Soffit of the nave**	**Damage post 1997 earthquake**	**Interventions**	**Damage post 2016 + Considerations on the response**	**Activate mechanisms post-2016 earthquake**	**Date of the Form A-DC for churches**
Castello al Monte in San Severino Marche	Via Castello al Monte	Domo Antico	Built 14th-18th century on the previous church of the 10th-11th century	Rectangular single nave	Bell tower in the facade plane with buttress base; gothic pinnacle shape	Façade:alternated rectangular white and rose stone units up to 8 m, then rectangular clay bricks.The church gate: squared stone	Masonry soffit with a square box configuration that covers the principal nave, masonry vault/dome in the apse area	N/A	N/A	Collapse of small portions of the masonry square box made with the technique of reeds and mortar+detachments and cracks in the extrados of the arches close to the facade(inside the church)and of those of the presbitery+lesions in the vault of the presbitery+vertical crack in the crown of the arch and in the entablature of a lateral chapel (it seems thatthe damage migrated from the above dome)	MEC 17,MEC18, MEC 19, MEC20	17/01/2017
San Severino Marche	Via Glorioso	Santa Maria del Glorioso	Beginning 16th century	Rectangular plan: 3 nave church. Columns separate the naves	Absecnce of bell tower, presence of bells in a belfry, regular clay brick pattern tessitura	Rectangular masonry bricks (exterior walls plastered while inside the church they are visible thanks to the cracks)	Barrel vault in the principal nave, groin vaults in the lateral naves, dome in the apse area	Damage in all perimetral walls with passing cracks, crack in the crown of the vaults and partial collapse of the dome	Consolidation of the walls; steel bars were inserted at the level of the drum of the dome and consolidation of the dome	Cracks below the rose window visible from the inside the church (activation of mechanism 2). Cracks in the vaults and triumphal arches of the lateral naves, transept and apse area. Shear cracks in the walls. The intervention of consolidation of the walls reacts quite well, helping the performance of the structure. Diffused cracks in the dome in correspondance of the openings and of the aisles at the base of the dome. Detachments in corrispondance of the arches located below the aisles.	MEC 2,MEC 7,MEC 9,MEC 11,MEC 12,MEC 13,MEC 14,MEC 16,MEC 17,MEC 18	07/05/2017
Dignano in Serravalle di Chienti	Frazione Dignano	San Lorenzo	Built 1800-1900 on the previous church of the16th century	Single nave with lateral chapels	The sacresty and the bell tower (built after 1997) are located in another area of the complex (not usual position)	Irregular stones and of different size (church)+masonry bricks for the belfry	Thin masonry barrel vault in the principal nave. Thin masonry barrel vaults in the lateral naves, groin vault in apse area, rebuilt after the previous collapse. Timber truss in the roof system.	Almost complete collapse of the roof, of the sacristy, of the presbytery and of masonry portions of the external walls closed to the bell tower. Widespread shear cracks in the facade. The active machanisms were: overturning of the facade, sliding and shear cracks. Transversal repsonse of the nave with overturning of the arches(presence of shear cracks at the base and at the crown). Vaults made of stones called "sponga"(typical stone of Terni area) of poor quality. Punching effect of the roof structure in longitudinal and transversal direction of the nave(absence of anti-seismic protection)	Inplane structural cross-bracing at the roof level; new secondary beams have been inserted in the roof structure. The walls were consolidated	Good response of the executed interventions, in particular of the roof structure. Inplane cracks in the facade with "V" shape. Good behaviour of the structural cross-bracing, pouching effect of the beams in the masonry walls (mec 19). Damaged thin masonry vaults(it seems they have been inserted later the 1997 earthquake; none info of damage post 1997). Good response of rebuilt portions of the walls. Some ligth cracks in the lateral walls and overturning of the gable	MEC 2,MEC 3,MEC 5, MEC 6,MEC 7,MEC 8,MEC 9,MEC 13,MEC 17,MEC 19,MEC 25,MEC 28	SC:23/02/17
ana in Serravalle	Località Acquapagana	San Salvatore	11th century	Single nave	Bell tower included in the structure of the church	3leaf walls (exterior:square stones;interior roubble stones)	Timber structure roof (no trusses)	Partial collapse of the bell tower, collapse of the gable, widespread shear crack in the plane of the facade; collapse of portion of the upper part of the lateral walls with portion of the butresses. Shear cracks in the plane of the triumphal arches.	Replacement of the heavy concrete roof with a timber light structure; rebuilding of partial collapsed of lateral walls, of the butresses and of the bell tower	Good response of the interventions. In particular the positive intervention in substituting the conscrete roof with a light timber roof. Pounding effect of the roof beams, some cracks are visible in the facade, visible from the inside of the church. Crack along the height of the bell tower (partially rebuilt after the 1997). Good response of the buttresses and lateral walls partially rebuilt after the 1997. Shear cracks in the facade recalling the discontinuity.	MEC 1,MEC 2,MEC 3,MEC 5,MEC 17,MEC 18,MEC 27,MEC 28	22/02/2017
Plestia in Serravalle di Chienti	Località Plestia	Santa Maria	10th century	Single nave+narthex	Absence of bell tower	irregular stones (walls)	Light timber structure in the nave+concrete roof in the apse area	N/A	N/A	Detachment of the facade (visible outside)+overturning of the gable. It seems that the walls in the apse area have a overturning behaviour in base of the provisional intervention set (to be confirmed). Damage in the vaults of the crypt already damaged (presence of supporting provisional structures). Lesions in the facade (inside) due to the pounding effect. Lesion close to the big opening in the apse area (maybe due to the presence of the concrete roof.)	MEC 1,MEC 2,MEC 3,MEC 5,MEC 6,MEC 17,MEC 18	23/02/2017
rravalle di Chier	Via Santa Lucia	Santa Lucia	1400;last transforma tion 1690	Rectangular nave+1 lateral nave	Bell tower included in the structure of the church	irregular stones	Truss timber roof+ low barrel vault made of reeds + vault/semidome in the apse	Between 1998 and 1999 the roof was replaced, tie rods to anchor the gable were inserted. Treatment anti-woodworm done on the roof beams and reed vaults. Maintenance intervention in the mortar of the exteranl walls and of the bell tower(calce romana+marmorite). Demolition a level of the annexed building in the north wall, rebuilt the roof of the annexed body. in the 60's the rose window was created in the facade+escavated the facade wall to insert an organ, a wall was cut to create lateral chapels. 3leaf walls (infilled material). In the 2002 stability test of the walls. Before the 1997 Crocifisso chapel was damaged in the pillar of the arch.	Consolidation of walls (groun injections) and secondary beams of the roof. Structural cross-bracing in the roof	Gable mechanism due to interventions of the 60's. Shear cracks in the apse area and lateral walls. Cracks in the vault of the aspe and in the triumphal arches. Good box-like behaviour helped by the interventions in the roof. Overturning of the facade. The bell tower suffered damage.	MEC 1,MEC 2,MEC 6,MEC 13,MEC 17,MEC 18,MEC 23,MEC 28	22/02/2017
Toelentino	Piazza Maurizi	San Francesco	13th century- Last transforma tion 1800 approxima tely	Rectangular nave	Bell tower included in the structure of the church	rectangular bricks	barrel vaults with lateral aisle + transversal arches in the central nave + semidome/vault in the apse area	N/A	glue-laminated beams in the roof system+ light steel closed beam+tie rods (emergency intervention)	Good structural response of the roof intervention. It contributed to the box-like behavior, avoided the overturning of the external walls and controlled the collapse of the arches and of the vaults of the central nave. Progressive damage during the seismic events both of the out-plane and in plane modes of the facade	MEC 1,MEC 2,MEC 3,MEC 5,MEC 13MEC 17,MEC 18,MEC 23,MEC 24,MEC 27	08/02/2017
Tolentino	Via Santa Maria 50	Santa Maria Nuova called 'La Tempesta'	13th century. Last transforma tion 1740 approxima tely	Central plan configuration with 4 chapels	Absence of bell tower	Bricks	thin masonry brick vaults in the chapels + dome in the principal nave	Closed after the 1997 earthquake until 2005. A new roof was built and as well as a double ring steel system around the drum of the dome	double ring steel system around the tombour of the dome	The intervention on the drum of the dome helped in the structural response, but in the same time facilitate the migration of the damage in the triumphal arches and vaults of the lateral chapels located at the base of the drum. Shear cracks in the lantern	MEC 5,MEC 8,MEC 9, MEC 13,MEC 14,MEC 15,MEC 17,MEC 18	08/02/2017

Identification the church				Materials & Structure				Information on earthquake damage and interventions				
Municipality	Address	Name of the church	Year	Plan Configuration	Other relevant info (i.e. facade-bell tower)	Construction material	Soffit of the nave	Damage post 1997 earthquake	Interventions	Damage post 2016 + Considerations on the response	Activate mechanisms post-2016 earthquake	Date of the Form A-DC for churches
Tolentino	Piazza del Ricovero	SS Crocifisso	16th century	2 nave church (Principal+lateral chapel). Rectangular plan	Absence of bell tower	Bricks	Barrel vault made of thin masonry bricks in the principal nave+pavilon vaults in lateral chapels+Thin masonry brick vault in the apse area+truss timber roof	N/A	Thin masonry brick vault built in the 18th century according to the baroque fashion. Introduction of an element of vulnerability. Absence of anti-seismic protection details	Absence of intervention in the vaults, consequently collapse of the entire vault of the nave+partial collapse of the vault in the apse area	MEC 6,MEC 8,MEC 13,MEC 18,MEC 23,MEC 24,MEC 28	07/10/2017
San Ginesio	Piazza Gentili 37	Collegiata di Sant'Andrea		Rectangular plan: 3 nave church. Columns separate the naves	Rectangular façade ("All'abruzzese")+Bell tower inside the structure of the church	Regular square stone+brick in the bell tower	groin vault made of bricks with vertical stacked bond+ octagonal dome in a lateral chapel	N/A	N/A	Cracks in the dome of the lateral chapel + shear cracks at the top of the pillars of the lantern. Partial collapse in correspondance of the crown of the groin vaults of the lateral nave and of the principal nave. Overturning of the facade caused by lack of anchoring with the transversal walls and facade "all'abruzzese". Tie rods in the central nave and in one of the two lateral nave, this aspect could have caused an asymmetric global behaviour of the structure.	MEC 2,MEC 3,MEC 5,MEC 8,MEC 9,MEC 11,MEC 13,MEC 14,MEC 15,MEC 16,MEC 18,MEC 25,MEC 26	N/A

APPENDIX
IC
THIN BRICK MASONRY VAULTS

In the last earthquakes that hit Central Italy, the 2009 L'Aquila earthquake and the 2016 Central Italy earthquake, thin masonry vaults have shown to be particularly vulnerable under seismic actions (Kaplan et al. 2010; Nunziata et al. 2017; Parisi and Sferrazza Papa 2017). Aiming at the preservation of this architectural element, largely common in Umbria, Marche and Abruzzo as a specificity of the local construction practice, the intention, here, is to trace the characteristics of these vaults.

CHARACTERISTICS AND GEOGRAPHICAL CHARACTERIZATION

The terms *tiled vaults* or *thin masonry vaults* identifies a specific construction technique. It consists of laying 'flat' bricks, following the curvature of the vault, there is the advantage that the construction process is fast and easy and, in some cases, without the need of centerings to support the construction in progress. This kind of vault can be found both with a single layer of bricks and with multiple layers, from two up to four. They can have bearing or decorative functions. The stiffness of this system is given by the construction technique itself and not by the thickness. An explanation on how these vaults work is treated in Guastavino (1892). Guastavino identifies two construction techniques for vaults: the *mechanical constructions* and the *cohesive constructions*. In the first case (Fig. 1), the constitutive elements are kept together by the gravity force and in the case of structure dismantling, the construction materials can be easily reused (i.e. the stones of Pyramids in Egypt or of Greek temples). In the second case (Fig. 2), the units are kept together by the cohesive force: the mortar and the bricks work as a unique mass; in this case, separating the different components without destroying the entire volume is impossible (i.e. Babylonian brick walls or vaults and cupolas of the Assyrian, Persian, Roman ad Byzantine empires) (Guastavino 1892).

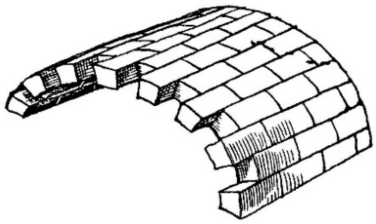

Fig. 1 The gravity system. A vault made of *voussoirs,* radially arranged (Collins 1968, p.49)

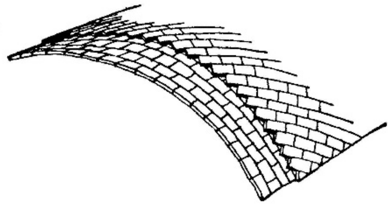

Fig. 2 The cohesive system. A vault made of 2 layers of tiles. (Collins 1968, p.49)

Moreover, in the same Essay, it is explained that, in the case of single layer vaults (Fig. 3a), the way of responding of the vault is comparable to the gravity system. Indeed, the single elements work as voussoirs with head joints. On the contrary, with multiple layer vaults (Fig. 3b), the vault is a cohesive system because the overlapped layers have stepped joints and, with the interposed hydraulic mortar, the structure works as a unique mass.

Fig. 3a Single layer vault (Guastavino 1892, p.49)

Fig. 3b Two-layer vault (Guastavino 1892, p.49)

The origins of this construction technique are still not clear. A long discussion, since remote times, has been carried out; for more details on this aspect Collins (1968) can be consulted. What is interesting to point out is the large use of the thin masonry vaults in the Mediterranean coast lines and how such an architectonic element was adapted in the use of materials and setting. The name of this practice also changes: in Spain *bóveda tabicada*, in France *voûte à la Roussillon* or *voûte plate*, in Algeria *rhorfas* and in Italy *volte a foglia* (Collins 1968). In France, these vaults take the name of the southern area, Roussillon, the area where this technique had a large diffusion. It is adjoining to Catalunya, the area of Spain where the *tecnica tabicada* was strengthened, thanks to the work of Rafael Guastavino. Whether Spanish builders were responsible for the spreading of this technique in France or it was an indigenous technique is still a debate, even if an interesting explanation is reported in Collins (1968). In any case, Rafael Guastavino was the person that, first, in Catalunya and, later, in America refine this technique (Guastavino 1892; Gulli 2006; Ochsendorf and Freeman 2010). Independently of the origin of this technique, what can be said is that this kind of vaults started to be largely used between the 18th and the 19th century. The reasons that could have favored the large use of this construction technique are:

- the fast and easy use of the plaster during the construction of the vault, without necessarily recurring to centerings;
- the versatile and adaptable characteristics of this construction technique.

- the resistance to fire, as Guastavino demonstrated making of this aspect a slogan for his company, Guastavino Fireproof Construction Company.
- in the case of the timbrel vaults, the great strength capacity of this multi-layer system.

It was demonstrated through experimentation carried out in Manhattan in 1901 (Collins 1968) and the durability of this multilayer technique was seen in occasion of the works at the Metropolitan Museum vaults (Collins 1968).

In Masi (1788), the presence of these vaults in France during the XVIII century is due to their stability and easy and fast construction. They were built with two layers if they were based in the upper level, otherwise with only one layer. The quality of the material was pointed out as an essential aspect for the efficiency of these vaults: bricks should be well fired and with the dimensions of 10x5 *digiti* to be set 'flat' using the plaster. The diffusion of this technique in France is confirmed by the manual entitled *Manière de rendre toutes sortes d'édifices incombustibles* (Espie 1754). After this publication, he was also invited in England to lead some industrial works.

In 1876, Guastavino presented at the Philadelphia Exposition some plans for the application of the technique for a building that he called the *tubular construction*; the title that accompanied the table was "Improvement in Public Health in Industrial Cities". Indeed, this technique was in accordance with the principles of hygiene and public health of that time. The "tubular construction" consisted in the use of flat bricks to build walls and floors.

In the Canadian context, in particular in Montreal, such technique was not found, except for one church over 108, and more similar to the American style (brick + cement mortar). The reason could lay on the large availability of timber as a construction material.

For what concerns this technique in Italy, depending on the area, it took different names: in the western part of Sardinia, in Alghero, such vaults were called *volte alla catalana*, in Central Italy *volterrane*, in Sicily *volte realine* o *alla Siciliana* (Frattaruolo 2000).

In Sicily, the vaults consisted of three layers of tiles called *tivuli* with a size of 2x10x20 cm set with plaster with the use of centring. In this geographical area, this technique also presents, as a constitutive part, counter vaults located above the tiled vaults to reduce the extrados filling material and consequently the weight of the entire construction (fig.4).

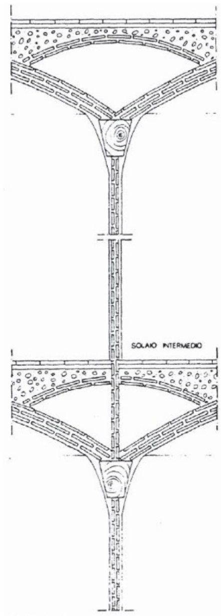

Fig. 4 Timbrel vault with counter vaults. (Frattaruolo 2000)

In some cases, ribs were added in the extrados of the vaults, which contribute to its stability (fig. 5).

Fig. 4 Photo of a timbrel vault with diagonal and longitudinal ribs (Fatta et al. 2017).

An example of a thin masonry vault typical of Central Italy is shown in (Giovanetti 1992; Cangi 2005). A load-bearing thin masonry vault is described. This vault (fig. 5) was built in 1880, according to the documents found in the Municipal Archive of Città di Castello. The vault was introduced in the occasion of a change of use of the building into a slaughterhouse. In the document, it is called *volterrana*, and it was made of bricks with a size of 33 x16,5x6 cm³. The thin thickness, which could have suffered instability, was compensated by arches in the intrados and filling incoherent material in the extrados. In the noble level of the same building, there are other thin masonry vaults but with decorative function, as those that can be found in churches which underwent interventions in the 18th century. These thin vaults were rigid in their plane due to a timber floor located above the one-layer vault, filled up to the haunches. This is a good solution to help to avoid the displacement of the bearing walls, but not always is possible nor present and its maintenance is rather difficult.

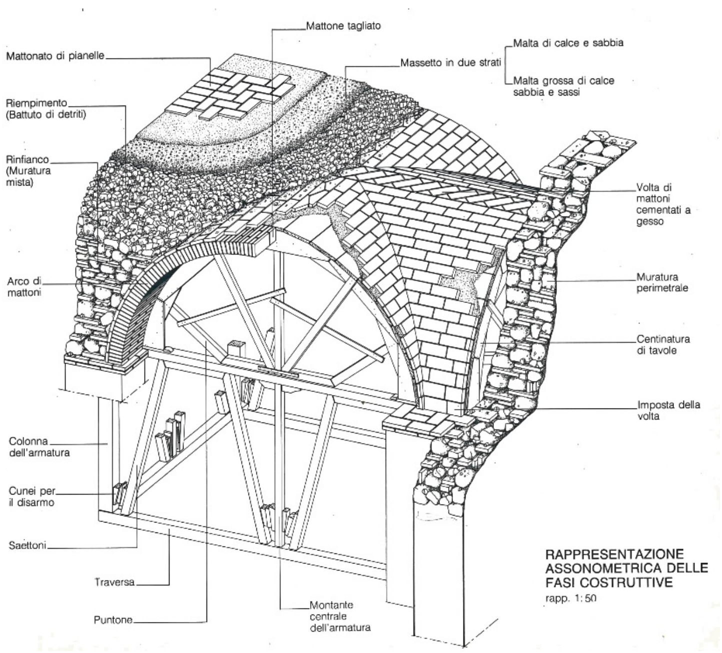

Fig. 6 Axonometric view of a typical load-bearing thin masonry vault of Central Italy (Giovanetti 1992).

APPENDIX II

INFORMATION ON THE INVESTIGATED CHURCHES IN EASTERN LOMBARDY

*The source of this information is (Superintendence of Brescia 2018).

Identification the church					Materials & Structure				Information on earthquake damage and interventions			
Municipality	**Address**	**Name of the church**	**Year**	**Position**	**Description of the church**	**Construction Materials**	**Construction Tecnhiques**	**Historical information**	**Damage post 2012 earthquake**	**Interventions**	**Post-earthquake safety intervention report**	**Interventions post-2012 earthquake**
Quingentole	Piazza Italia	San Lorenzo Diacono e Martire	1751-1754	Connected with a building on one side	Unique nave obtained by the intersection of 2 ellipses+4 lateral chapels+ presbitery +apse. Three entrances in the façade and, along the lenght of the nave, one per side close to the presbitery. There is a bell tower 38 m high with a square plan. The rythme of the nave is given by the repetition of pilasters and capitals, an entablature separates the wall of the nave from the vault which covers the nave. The entablature is composed of a projecting cornice, a frieze, and a lintel The barrell vault, made of thin brick layer, of the nave has ribs and some grains in correspondence of the lunettes of the windows There are 4 windows with lunettes in correspondence of the lunettes along the nave, one in the façade wall, two in the presbitery and three in the semidome of the apse, decorated with ribs The presbitery is covered with a barrel vault decorated with ribs as that on the nave Two doors connect the presbitery on one side, with the sacresty and, on the other side with The lateral chapels are covered with decorated barrel vaults On top of the entrance there is a timber choir The façade is soaring respects to the roof. Pilasters and columns enrich it. It is composed of two orders, the first is divided in three parts and the second corresponds to the size of the nave Four buttresses with scrolls per side caracterise the lateral walls of the nave at the exterior The bell tower is characterised by four orders: one up to the level of the height of the lateral chapels, one up to the height of the presbitery and the other two that describe the space for the clock and the bell tower.	The material comes from the old church and the 2 lions comes from the cathedral of Mantova that was going to be rebuilt in that period The façade is covered with a white plaster contrary to the lateral walls and the bell tower that clearly sow the pattern of the brick wall. The church is made of bricks	The walls of the entire church have at least two leaves of bricks in their section The vaults are made of brick with vertical stacked bond. They have some ribs in the extrados	Built as copy of the church of Revere Built in the same place of a precedent church of the XV century that was demolished and rebuilt in front to the summer house of the Bishop of Mantova, Antonio Guidi di Bagno	MEC1: The longitudinal effect of the earthquake has caused a pounding effect of the roof in the façade wall weaken by the presence of the opening in the second order and the belfy part of the façade. The pinnacles present in the upper part of the façade collapsed. Mec2: The belfy façade together with the stifness of the masnory along the height and the presence iof a concentrated mass (the cross) on top of the façade have been caused of a rotation along the cornice of the gable. Shear cracks in correspondance of the openings of the nave, the apse,and the presbitery Diffused cracks in the vault of the nave, in particular in correspondance of the grains, of the chapels and of the presbitery and the apse in the centre and in proximity of the openings Severe damage of the triumphal arc probably due to the stifness of the system arc-pillar of the triumphal arc, to the inadeguate thickness of the arc or of the shear walls, and to the absence of a tie rods. The cracks are localized in the key and at 30° of the arc. The presence of the shear walls due to the restriction of the dimension of the nave has reduced the in-plane action avoiding the rotation of the pillar of the arc.	**Description of suggested structural works (A22.1 form of damage)** Structural assessment and consolidation of the buttresses Crack repair and consolidation of vaults (close to the triumphal arc and presbitery) Consolidation of the annexed bodies to the presbitery Check efficiency tie rods Check tie rod of the triumphal arc (capochiave is missing) Suggested by the superintence a pair of longitudinal tie rods along the principal nave Repairing of cracks and missing portion of paintings **Historical interventions** The church was enlarged and moified in 1776 and 1886. In the 90's, the vaults were retrofitted and the roof remade In 2006, retrofitting of the masonry walls	To avoid the overturning in the months immediately later the earthquake, a provisional intervention that is going to become permanent was been performed. This is a UPN frame, fixed in the back side of the façade wall (lofty part) and in the area under the roof. It was anchored to the façade with wall anchors. This is accompained with longitudinal tie rods and transversal ones that connect the façade with the longitudinal walls.	Consolidation of the triumphal arc with insertion of steel elements. Injection of grout, insertion of cast iron bars and elicoidal dry bars consolidation of the arch between the nave and the nartex with injections Consolidation of the damaged vaults of the nave, presbitery, apse, lateral chapels and sacresty with injections, inserting chocks and laying fibroreinforced grout with a net in the extrados Consolidation of the arc in the lateral walls of the nave that give access to the chapels with injections and elicoidal dry bars Consolidation of the walls of the sacresty with 'cuci-scuci' Consolidation of the trusses with rafter and fixing elements Installation of shelves in correspondence of the truss connections with the masonry and insertion of a tie-rod with cable-heads for the connection of the trusses to the walls Insertion of metal tie-rods next to the timber trusses that show critical statical performance Connection of the anti-overturning structure of the gable, inserted immediately after the earthquake, with the new bracing roof structure 'Insertion of an in-plane bracing roof structure
Quistello	Via Cesare Battisti 22	San Bartolomeo Apostolo	1730-1745	Connected with a building on one side	Tipycal orientation of the apse towards the east The church has a Greek cross plan with annexed volumes. It has a central cap and arms covered with barrel vaults, sustained by four composite pillars. A thick cornice separates the vault from the pillars. At the corner of the greek cross four symmetrical space, together with the arms of the cross, create two lateral naves. A system of arches defines the passage between the different type of covering.In the right nave an opening gives access to the canonic house.The presbitery is two steps above the nave and on one side it is attached with a chapel (left), and on the other side with the sacresty (right). In both cases, the access is through the apse. The façade has three entrances reflecting the internal subdivision.Above the lateral entrances there are two niches.Some pilasters emblish the façade and give it rhythm. A projecting cornice and entablature sepate the first from the second order of the façade. At the second order the shape of the façade is characterised by a pair of simplified volutes that harmonise the difference of dimension from the first to the second order. A central closed big opening is in the middle of the façade at the second order level. On the top of the gable, an acroterion and a marble object support the cross. The bell tower is indipendent from the rest of the church (it is not standing vertical already before the earthquake) There are windows at the second order level in the lateral walls of the church. Two orders of openings, one per side. A closed opening in the façade.	The bricks for the construcion of the church come from the local castle and the ancient church built far away The façade is covered with a white plaster contrary to the lateral walls and the bell tower that clearly show the pattern of the brick wall. The church is made of bricks		Arch.Giovanni Maria Barsotto designed the church	**Previous damage in the history of the church** In 1888 the Ing. Carlo Barboglio reported a ground feilure with the lowering of the four central pillars and the west wall (10 cm lower than the east). It seems that since the time of construction of the church a north pillar lowered, causing deformations of the above-structure, especially to the arches connecting the pillars, and to the cornice. To try to solve the problem two buttresses were built on the north side.Still the structural problem continue, denouncing that such solution was not sufficent to solve the problem. In the same period the roof was repaired. In 1900 the Ing. Comunale pointed out the inefficiency of the foundation in the first pillar on the left entering the nave, but there are not proven data of that. In the '20-'30 important repairing works for the roof were done, in fact it seems there are two leaves of roof, one with brick tiles and the other one in mixed concrete and tiles.In the 50's partial substitution of portion of structural elements, bricks, and tiles. **Post 2012 earthquake** **After the 20th May shock** Activation of the kinematic mechanism of overturning of the gable. Collapse towards the interior of the church of the marble element, located on top of the façade supporting the cross, at part of roof behind the façade and the against-façade vault. **After the 29th May shock** This second shock wosen the damage of the agaist-façade vault and damaged other vaults causing secondary damage inside the church The collapse of the heavy decorative element behind the façade had further implications, as the collapse of the organ and of the internal frames of that area, actually increasing the collapsed zone The apse had numerous inclined cracks, mainly due to a torsional movement on its axis, that can be observed in the internal walls of the apse. The arches of the longitudinal walls of the central nave were damaged in correspondance to the chapels. Those cracks on the arches appeared extended up to the vaults of the central nave, with some collapsed parts.	**Description of suggested structural works (A22.1 form of damage)** Repairing of the damaged roof Repairing of the damaged vaults Repairing of the cracks in the walls Insertion of tie rods in the façade to avoid the overtuning Insertion of new tie rods where needed and check of the existing ones		Rebuilding of the timber roof structure, using appropriate dimensions and connections. Insertion of a timber diaphragm,made of multi-layer panels, paying attention to the connections with the masonry structure The collapsed portions of the vaults are rebuilt in solid bricks. The cracked vaults are strengthened with lime mortar and steel wedges, completed with binder strips of polimeric materials (PBO: Poly-p-Phenylene Benzobisoxazole)on the extrados The walls are rebuilt with the same type of bricks where portion are collapsed. Mortar with pieces of bricks are used to infill cracks in wider cracks of the walls or steel wedges are inserted The efficiency of the existing tie rods is checked and new ones inserted where needed Check of the foundation to see if any strengthening intervention is required

Identification the church					Materials & Structure				Information on earthquake damage and interventions			
Municipality	Address	Name of the church	Year	Position	Description of the church	Construction Materials	Construction Tecnhiques	Historical information	Damage post 2012 earthquake	Interventions	Post-earthquake safety intervention report	Interventions post-2012 earthquake
San Giovanni del Dosso	Via Roma 11	San Giovanni Battista	1616	Isolated	The church is directly at the border of the street witoout a square in front of it. The façade is composed of two orders with the lower one that reflects the three-nave distribution, and the upper one that has a reduced dimension corresponding to only the central nave. The volutes have been added at the end of the construction of the church at the end of the XVIII century together with the lateral entrance doors. A projecting cornice and entablature sepate the first from the second order of the façade. At the second order of the façade the church shows two symmetrical niches and a rose window close to the cornice which separates this order from the ground. The lateral elevations have two level and has a rythm due to the buttresses that reflect the internal subdivision with round openings in correspondance of the grains (unghie) of the vaults It has three nave realised in the XVIII century including existing chapels The central nave is covered with five barrell vaults of thin layer bricks distiguished one from the other with arches: while the first, the fird, and the fifth are simple barrel vaults, the second and the fourth also show the lunettes in correspondance of the openings and they are made of bricks laid in their lower dimension ('a coltello'). The lateral naves are covered with groin vaults. The presbitery is covered by a barrell vault made of bricks laid in their lower dimension ('a coltello'). On the right side of the presbitery, there is the sacresty while on the left a chapel. The apse shows the bricks without any plaster layer as the rest of the church. The arches between one vault and the other have a width of 70 cm and a thickness of 30cm and they have a tie rods at 1/3 of the height of the arc, connected to the external walls. Moreover at the impost, the arches are reinforced with 'frenelli' along the complete length. At he border of the arches they are realised with notches that facilitate the connection with the vaults. The vaults have not any ribs, usually made of bricks laid in their smaller dimensions ('a The arches support directly the beams of the roof. The vaults of the lateral naves are groin vaults made of thin brick layer with the same rythm of the vaults of the principal nave. They are separated with arches (made of two layers in its section='a due teste') showing the same techinique of the principal nave. Any tie rods were present in the vaults of the lateral naves. The structure of the roof is realised, both for the principal and lateral naves, with purlins on top of which the seconday timber beams are supported according to the inclination of the roof. Some quarry tiles constitute the plane of the roof. Some of them were substituted with wooden pannel or brick hollow flat blocks. In the central nave, new timber beams were attached to the deteriorated existing ones The plane of the roof of the lateral naves is mostly constituted by brick hollow flat blocks. The structure of the roof of the apse has a principal structure some trusses and radial beams which support the perlins and secondary beams, and the plane of the roof is realised with quarry tiles. Some steel elements have been noticed in the connection between the rafter and chord	Vaults made of thin layer bricks ('in foglio')		This church was built where there was a Matildic church, romanic style, that for its compromised conditions was demolished in the XVII century. Because of its compromised structural health, the bell tower , previuously annexed to the church between the nave and the presbitery, was rebuilt 7 far in the west side between the 1963 and the 1970. From an inventory dating back to the 1728, it is known that the church had one single nave, covered with a vault at the choir level, and four lateral chapels. Probably when the church had a single nave the two square areas symmetrically located respect the principal entrance were two openings. The second nave, on the left side, was added between the 1682 and the 1724. Two renovations of the façade are known: one on 1712 and one on 1853. The inventary of the 1793 reports that the church has three naves, on the left three chapels, the bell tower, a secondary entrance. On the right side, there are four chapels. 1872 the semicircular choir was built as enlargement of the presbitery 1925 the roof was repaired, as the plaster of the façade and of the bell tower In 1920 a chapel on the left side of the apse was built to be dedicated to the war dead. In the right side the sacresty and the bell tower appear in a cadastral map of the time. In 1963, a survey on the health of the structure pointed out that the level of the ground waterreached the 1,77 m and that the foundation and walls of the bell tower made of bricks are made in a good state of conservation except for the mortar that is deteriorated. The bell tower is inclined towards the courtyard between the town hall and the canonic house (0,82 m). The church showed vertical cracks due to the movement of the bells of the bell tower. The roof is damaged because of water infiltration, at the same way the gable is damaged by the water. Some cracks in the façade and in the riight side of the entrance in the lateral wall. In consequence of such state, to play the bells was forbitten, the sacresty was closed,the use of the left nave was forbitten and the dangerous façade was imposed, the roof and the rain system of the roof was checked. In 1965, the parish priest asked for the authorization for the demolition of the bell tower due to the continuous structural problems. In 1968, the demolition was authorized and the bell tower was documentated in case of a rebuilding in the future where it was, as it was. The new bell tower is realised with a concrete structure, on top of a reinforced concrete foundation made of ground pillars. The walls were realized with bricks and covered with plaster to have an appearence as the demolished bell tower. In 1989, infilling of the cracks in the mansory walls, in the arches, and in the groin vaults, restauration of the cornices with lime mortar and other non-structural decorations.	**Reported state of conservation for the roof** Some beams of the roof are damaged by the water infiltration Some beams are reinforced with inserted timber elements Some nodes, in particular between the chord and rafter, seem not to be effective The supports of the purlins are not always well locked **Post 2012 earthquake** Collapse of the vaults (barrel vault, barrel vault with lunettes) of the first and second spans of the nave. Detachment of the circular walls of the apse from the presbitery, as well as the associated vaults. Portion of the building realised in different moments. **After the 20th May shock** Detachment of the façade from the longitudinal walls (bigger at the top of the façade, principal nave, than at the bottom,lateral nave) and cracks in the plane of the façade, in particular in the gable area. **After the 29th May shock** Collapse of the gable at the level of the second cornice, causing the consequent collapse of the attached portion of the roof structure and of the vault of the first span of the nave. This was caused by the absence of connection between the façade and the roof, absence of ring beams and stiffening in-plane structure in the roof. Moreover, the stiffness of the façade wall and the presence of the opening at the centre of the façade have falitated the activation of the overturning of the gable (MEC 3). The detachment of the façade from the longitudinal walls increases and the façade shows an overturning of 10 cm, facilitated by the absence of longitudinal tie rods and not efficient connection betwen the façade and lateral walls. Some cracks in the plane of the façade are more visible, in particular a wide crack between the lintel of the principal portal and the upper rose window. Complete collapse of the vault of the first span (simple barrel valt), and of the second (barrel vault with lunettes), caused by the overturning of the façade and of the londitudinal and transversal direction of the earthquake. Cracks and partial collapse are also present in the other vaults. The system of arch-pillar between one vault and another response with a good answer to the earthquake and did not suffer consisttent damage. The first vault close to the façade on the left side suffered a severe collapse of half of the vault. Some cracks following the direcction of the arc of the groin vaults were observed. Severe cracks were observed in the longitudinal walls of the central nave and in the walls close to the triumphal arc and they occupy the portion between the arcs in the lateral walls of the principal nave and the upper windows. Wider cracks are visible in correspondance of the eave. Inclined cracks are visible in the lateral walls denouncing the poor resistence capacity of the wall and of the poor quality of the mortar. A severe crack of detachment is visible in correspondance of the apse-presbitery both at the vault level and in the walls. This is probably due to two different construction phases of the walls and not-well connected.The first portion of the vault of the presbitery was realised with brick laid according their smaller dimensions ('a taglio') while the last portion and the curve part was realised with thin layer bricks ('a foglio') with ribs with two bricks in the section ('a due teste'). Thermographic investigation to check the condition of the masonry walls. They did not show any difference compared to whaat it is visible to the eyes.	**Description of suggested structural works (A22.1 form of damage)** Rebuilding of the collapsed vaults Intervenions of 'cuci-scuci' Rebuilding of the gable Injection and repairing where is required check of the timber structure of the roof, restauration and repositioning of the cross on the top of the façade Insertion of longitudinal tie rods Insertion of a light stell ring of connection between roof and walls Consolidation of the thin brick vaults of the central and lateral naves The bell tower was not inspected because inaccessible, but from the exterior visible cracks a more accurate visit was required		Rebuilding of the collapsed gable with the fallen bricks and new ones of the sam dimensions laid recurring hydraulic mortar. The pilasters, existing in the back of the façade, are reinforced adding a brick layers in the portion standing above the roof. Four vertical metal tie rods are inserted in the masonry wall of the façade. They are connected inside the existing mansory and post-tension at the top of the gable. In correspondance of the cornice in the layer of the mortar between bricks unidirectional iron fibers are inserted and the same reiforced system is put in the upper part of the gable following the shape of the inclined lines of the border of the gable. The deformation of the façade, compromised in consequence of the overturning mechanism, was restored. Then the façade is connected in the interior of the church at the line of the cornice and of the eaves ('gronda'), and with the rebuilt against-façade arc. The cracks in correspondace of the connection façade and lateral walls are refilled. Finally the façade is connected with the inplane roof stifenning after its realization. **Interventions in the roof** Substitution of the deteriorated timber elements, the purlins are fixed with the rafter, the connection between the rafter and chord is guarantee with the insertion of a metal tie rod. A rigid ring beam is realized at the top of the walls of the church to connect the façade, lateral walls, apse and roof, where a timber in-plane diaphram is realized. The walls of the church are reinforced at the level of the roof with unidirectional steel fiber in the lines of the mortar between the bricks. A metal plate connects the lateral walls with the timber stiffening plane, added in the roof. The principal beams of the central nave are laid directly on or through small pillars on top of the principal arches of the nave made of bricks. At this regards, a wall connecting the the arches to the plane of the roof, the longitudinal walls is built in correpondance of each arch of the principal and lateral nave of the roof. The timber plane roof is connected with vertical bars to the new walls.The new added walls are covered with reinforced plaster. **Interventions in the vaults** The damaged thin brick layer vaults are retrofitted in the extrados with a thin layer of plaster reinforced with glass fiber and compatible with the masonry.The small collapsed portions are rebuilt and the glass fiber reinforcement is laid in the principal axis of the undamaged vaults. The thin brick layer vaults completed collapsed with the earthquake are replaced with faulse vaults (being anti-economic its realization) realised with a metallic structure that supports the plaster of the vault. The configuration is the same of the previous one (barrel vault with lunettes). In the apse at the level of the extrodos of the arc, a steel ('controventamento di piano') **Interventions in the longitudinal walls** A system of tie rods at the level of the internal cornice of the nave in the principal nave and apse, anchored in correspondance of the arches, is realised. At the extremity, the anchorage system is hidden in the plaster. When the cracks in the walls have wide dimensions, the missing portion is rebuilt with the 'cuci-scuci' technique using bricks and mortar. On top of the arches at the border of the principal nave and in the principal cracks of the apse, the cracks are infilled with bricks fixed without mortar, but with elicoidal steel bars. Repairing of the smaller cracks with the insertion of timber wedges and/or iron and inection of grout until rejection. For small and isolated cracks, injections of a mixed solution of sand, lime, and marble powder is put.

Identification the church					Materials & Structure				Information on earthquake damage and interventions			
Municipality	**Address**	**Name of the church**	**Year**	**Position**	**Description of the church**	**Construction Materials**	**Construction Tecnhiques**	**Historical information**	**Damage post 2012 earthquake**	**Interventions**	**Post-earthquake safety intervention report**	**Interventions post-2012 earthquake**
Borgofranco sul Po, Loc. Bonizzo	Via Ibardini 1	San Giacomo Maggiore Apostolo	1789		The church has three naves, result of modifications along the years. The principal nave is constituted by four spans separated by arches (with tie rods at 1/3 of the height) and covered with barrel vault with lunettes made of thin brick layer ('in foglio'). The vaults of the presbitery and of the apse are as well made of thin brick layer ('in foglio'). The lateral nave in the right has the same distribution of the the principal nave and it is covered with groin vaults separated by arches. The lateral vault in the left has smaller dimensions than that in the right with the same configuration of the other two naves and hosting altars. The timber trusses of the roof are supported by the lateral walls of the nave. On top of the trusses a system of secondary timber beams supports the roof plane made of hollow flat blocks.	Brick masonry walls, arches and triumphal arc		It was built in the second half of the 1500 and strongly damaged in 1538 by a flood of the Po river, rebuilt in a safter place in 1530. The church was transformed along the years: at the beginning it has a unique nave with three spans and an apse, and the bell tower. In the second half of the 1700, the church building includes the church, the sacresty, the parish house, and the cimitery. From the inventories of the end of the 1700, the church has two naves with different dimensions, the principal one and one on the right constituted of two chapels. The roof of the church was made of timber trusses with secondary beams and hollow flat blocks joint with mortar. On August,9 1785 the church was distroyed by a storm and rebuilt with the canonic house with an added span to the principal nave, two chapels to form the left nave between the 1788 and 1789. In the new church the principal nave was covered with a thin brick laer vault. Durinf the II World War the right lateral nave was built. In the 1951 the church was completely restored with the rebuilding of the right nave and the restoring of the façade as it appears today.	The principal mechanisms are: The longitudinal response of the arches and of the lateral walls; The feilure of the walls and vaults of the lateral chapels; Cracks in the triumphal arc and vault of the presbitery. **The longitudinal response of the arches and of the lateral walls** The arches in the lateral walls of the principal nave are cracked in proximity of the key, crossing the cornice up to to the above openings. The cracks also continue in the lintel of the windows and reaching the roof eaves. The cracks are visible botbh at the interior and exterior of the church. **The failure of the walls and vaults of the lateral chapels** Small cracks in the vaults of the lateral nave. The cracks have wider dimensions in the first two chapels of the left side, in paarticular in the connection vault and longitudinal walls, and in the walls close to the longitudinal walls of the principal nave. The hypothesis is that the craks already existed before the eathquake and grow in consequence of it. Indeed, they belong to a later building phase, the 1951. The walls, vaults, and roof of the chapels did not show any damage. **Cracks in the triumphal arc and vault of the presbitery** The triumphal arc is cracked in the key (crossing the section it is also visible from the roof) and other cracks appear in the walls close to the triumphal arc. These cracks extend themselves in the vaults with lunettes closeby. The triumphal arc also got the sollicitations of the bell tower standing attached on the left. The central crack in the triumphal arc also extend itself in the thin brick vault of the presbitery (always crossing the section and visible from the roof).	**Description of suggested structural works (A22.1 form of damage)** A transversal tie rod in correspondance of the triumphal arc and infill of the cracks are recommended after the post-earthquake survey. As well as the check of the foundation in correspondance of the left lateral nave. The repairing of the principal and secodnary structure of the roof is recommended in the central nave. Insertion of the longitudinal tie rods in the principal nave and of the transversal ones for the lateral naves. If exending the tie rods in correpsondance of the triumphal arc including the structure of the bell tower is to be evaluated. The failure of the nave in the left is pointed out, due to the proximity with the river Po. Afrer the II World War, two spans of this nave were rebuilt because destroyed by bombs. The roof was rebuilt in around 2000		Two longitudinal tie rods are inserted at the roof level at the impost of the timber beams. The tie rods are anchored in the façade and back elevation with root anchorage to be less invasive from an estetical point of view. Along the length of the nave the tie rods have intermidiate anchorage. At the roof level, a transversal tie rod is put back to the façade and in correspondance of the triumphal arc with plate anchorages. A tie rod is inserted in the triumphal arc at 1/3 of the height. It has a rectangular section 100x10mm in the visible portion and round with a section of 36 mm for the part hidden in the masonry. Two longitudinal tie rods are inserted at the cornice level in the interior of the nave (at 4,50m). The plate anchorage are hidden in the façade under the plaster. An intermidiate anchorage to linit the length is inserted. The vault od the presbitery and that close to the triumphal arc were repaired and retrofitted with FRP in the extrados because showed a more extensive damage. The choice was due to the way they were built with hollow flat blocks and with a thin layer of cement on top. Infill of mortar was done in the cracks of the walls and of the vaults less severe. The cracks in first two chapels of the left side the cracks between the longitudinal walls of the principal nave and the vault and the annexed vertical walls are evident and seems worsen after the earthquake. It was restored with mortar the join executed before to connect these two portions of the church structure of different periods.
Borgofranco sul Po	Via Roncada 29	San Giovanni Battista	1616		The church in neoclassical style has three naves with added chapels The timber trusses of the roof are supported by the lateral walls of the nave. On top of the trusses a system of secondary timber beams supports the roof plane made of hollow flat blocks. The church is composed of three naves: the principal one covered with a coffered ceiling with a timber structure hang with timber elements to the chorns of the timber roof trusses. The vaults of the lateral naves, the presbitery, and the apse are realised with a timber spar and reeds. The bell tower is attached to the church structure in the north close to the presbitery area.	Brick masonry walls, arches and triumphal arc		The small church was buuilt in the old part of Borgofranco del Po close to the Po river in 1616 as oratory dependent from the church in Bonizzo, that expecially in winter time was difficult to be reached. In a second moment, the church was enlarged for the needs of the community. In 1896, the municipality economically supported the restoration of the canonic and other small works. After the 1900, works on the elevation of the bell tower, adding the lead dome, and the arrangement of the clock. The works in the bell tower ended on August 1929 together with a restauration of the façade and relative cornices.	A previous damage pointed out is a broken beam of the roof structure. This is probably due to the umidity and micro-organisms. **Due to the 2012 Earthquake** Longitudinal resposnse of the arches and lateral walls; cracks in the triumphal arc and vault of the presbitery; cracks of the semicircular apse; cracks in the vaults of the lateral naves, in the walls and in the lateral chapels **Longitudinal resposnse of the arches and lateral walls** The arches in the lateral walls of the principal nave are cracked in proximity of the key, crossing the cornice up to to the above openings. The cracks also continue in the lintel of the windows and reaching the roof eaves. The cracks are visible botbh at the interior and exterior of the church. The dimensions of the cracks have wider dimensions in the first and last arc, probably due to respectively the façade and the bell tower movement. The movement of the bell tower also caused cracks in the lateral wall close to the triumphal arc that gave access to the presbitery. **Cracks in the triumphal arc and vault of the presbitery** The triumphal arc is cracked in the key and in the close walls at its sides. The craks extend themselves to the close vaults with lunettes. **Cracks of the semicircular apse** The semicircular apse walls showed vertical/arcuate cracks in the upper portion crossing the circular openings. The cracks cross the section and are visible at the interior and exterior side of the walls. **Cracks in the vaults of the lateral naves, in the walls and in the lateral chapels** The vaults of the lateral chapels were slightly damaged. They are made of 'incannuciato'			Two longitudinal tie rods are inserted at the roof level at the impost of the timber beams. The tie rods are anchored in the façade and back elevation with root anchorage to be less invasive from an estetical point of view. Along the length of the nave the tie rods have intermidiate anchorage. At the roof level, a transversal tie rod is put back to the façade and in correspondance of the triumphal arc with plate anchorages. The cracks of wider dimensions in the principal arches and vertical walls have been infilled with steel or timber chocks and subsequent grout injection compatible with the traditional masonry. Substitution of the damaged timber beam of the roof.
Ostiglia	Via Vittorio Veneto 55	Assunzione della Beata Vergine Maria	1889-1896		Two geometrical figures create the architectonic shape of the church: the square, repeated 7 times designs the nave with a cross shape, the square divided by two determines the dimension of the lateral naves and the isosceles triangle for the heights of the church. The roof of the principal nave is composed of: 8 timber trusses with regular span in the first 3/4 of the principal nave, 3 timber trusses in the apse up to the beginning of the curve part; 3+3 timber trusses in the transepts; 4 timber trusses in the square area of intersection between nave and transept; 2 timber trusses in the same area but laid following the diagonal of the square. On top of the trusses, some purlins are laid that support secondary beaams and terracotta tiles (2,5-3,00 cm). The trusses are anchored in the masonry and reinforced in the connection with the walls with timber shelves anchored with metallic elements. The connection between rafters, post, strut are realised with timber pegs. The structure of the lateral naves for the	Bricks for the walls (35 cm thick with three heads) Stone for the columns Reeds for the faulse vaults		The project of the church was given to Ing. Pietro Saccardo di Venezia in 1889. The bell tower and the small chapel in the right side were built in 1927. The chosen style is the Lombardy or romanic style.	**Post 2012 earthquake** The earthquake put in evidence the weakness of the structure already observed during the construction: the light reed vaults and the reduced section of the supporting walls (just 35 cm and high). Compromised by change of temperature and the age, the reed vaults and the gypsum ribs suffered damage and collapse of some portions. Craks were pointed out in the timber structure of the vaults and in the decorated plaster. The shocks caused sollicitations on the trusses in particular in those laid diagonally in the square of intersection between transept and nave, breaking the pillars in the corners, each of them supporting three trusses. Cracks in the section of the ribs made of plaster of the reed vaults. Cracks in the rose central of the naves made of plaster with risk of collapse. Longitudinal cracks in the ribs made of plaster of the principal arches of the principal and lateral naves. Detachment of the vaults from the external walls with severe cracks and risk of collapse of portions of that. Shear cracks in the masonry walls and in the pillars of the square connection between transept and nave. Degradation of some roof beams caused by water infiltration. From the inspection of the extrados of the reed vault a	**Description of suggested structural works (A22.1 form of damage)** Crack repairing Reinforcement of the anchorage of the timber trusses Longitudinal tie rods and in-plane diaphram at the roof level No damage was observed in the bell tower		**Cross section ('struttura reticolare') at the roof level for the principal nave, transept, apse** Steel plates to border the extremity of the 4 pillars of the square area of connection transept-principal nave and shaped to host the tie rods used to anchor principal nave, apse and transept. Steel structure for the connection walls-trusses and lonfgitudinal tie rods. Stiffening beams to connect the gable **Consolidation of the lateral nave with metallic elements and wooden pannels X-LAM.** Connection with UPN at the support points of the trusses to the masonry walls. Steel plate to connect masonry-roof of the lateral nave. Steel plate ('piatto a tralicio') to connect the cornice of the eaves with the roof of the lateral nave. X-LAM wooden pannel with a thickness of 6 mm laid on top of the existing purlins. **Consolidation of the reed vaults in the principal and lateral naves, transept, apse (extrados, intrados, ribs and rose elements)** Retroffiting of the reed vaults with jute fibers and insertion of steel connections with nut and washer. Plaster in the extrados of the reed vaults and substitution of the deteriorated timber elements.

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					roof is formed with half trusses and on top of that the system of the roof has the same layuot of the principal nave. The principal nave is covered with arches made of palster and gypsum attached to the reed vaults ('camorcanna') supported by a timber structure. Round arches supported by pillars characterise the principal nave as the lateral ones. These last also have tie rods. The façade is covered with plaster while the lateral walls show the brick pattern.				retroffitting action was required.			Insertion of metallic spikes in the intrados of the reed vaults. Consolidation of the rose elements from the intrados and extrados. **Consolidation of the external pinnacles** Infill of cracks, treatement of the surface Insertion of tie rods **Hoop of the apse** Insertion of steel strips in correspondance of the apse at the level of stell plates, set to connect the stiffening steel elements with the masonry **Localised interventions of consolidation** Crack infill inserting in the layer of mortar UHTS steel fibers and chocks to lock the retroffiting system to the masonry. Consolidation of the timebr elemets of the roof with steel elements. Consolidation of deteriorated portion of the timber elements with resin. Substitution of a roof beam in the lateral nave.
Revere	Corso Italia	Annunciazione della Beata Vergine Maria	1400 1750 - 1775 (new church bigger and facing the principal street in Revere)	Historical centre	The church has a curve façade with two orders separated by a cornice. An additional cornice separates the second order from the gable. The elevation is characterised by a series of niches and a big central opening in the second order, and three entrance doors in the first. The church has a unique nave made of three spans, a presbitery and an apse. The pilasters describes three small chapels per side. The series of spans are separated by arches and covered with barrell vaults with lunettes as well as the presbitery and a semidome in the apse.			In 1479, the priest Corsini was elected to manage the construction of the church. The complex included the church, the Carmelitan monastery, the bell tower, the cimitery and a hospital, and a small host place for pilgrims. In 1520, the church was consacrated. The bell tower, as it is today, was built by the Carmelitani between 1604 and 1607 after that the previous one was destroyed. 1750-1775 in the same area, but bigger and facing the principal street of Revere was built. The bell tower was left isolated. Between 1791 and 1793, two secondary doors were opened. In 1944, a bomb destroyed the sacresty. Between 1999 and 2000, interventions on the roof, retrofitting of the masonry walls and fo the buttresses were performed.	Two pots in the top of the arcuate façade collapsed, damaging a portion of the roof of the principal nave. Two crakcs ('passanti') in the back side of the gable are visible in the portion of the masonry that support the central cross.	**Description of suggested structural works (A22.1 form of damage)** To evaluate the insertion of tie rods. Consolidation of plaster. Consolidation of the triumphal arc. Consolidation of the arches of the lateral naves, left and right corner of the altar area. Check of the existing tie rods. The bell tower, the roof, and gable area could not accessed.	The sacresty was declared accessible. Safety intervention in the triumphal arch and to avoid the collapse of plaster from the vaults. Check of the existing tie rods.	Infill of the cracks with mortar and timber chocks from the extrados. Realization of a steel structure of counterthrust above the triumphal arc attached to the wall standing between the triumphal arc and the roof at roof level. Reinforced injection at the intrados of the triumphal arc with elicoidal bar and injection of grout. Insertion of a steel truss beam ('travi reticolari in acciaio') to the existing concrete beam. The steel truss beam is connected with tie rods realized at the roof level. Realization of 'frenelli' made of Poroton brick to connect the vault of the central nave with the façade wall at the roof level. Reinforcement of the existing tie rods of the bell tower with insertion of a new system of anchorage.
Bagnolo San Vito - Loc. San Giacomo Po	Via Priore 38	San Giacomo Maggiore Apostolo	1906 - 1909 1953 (realization of the bell tower)	Urban centre	The church has a single nave with three spans covered with groin vaults made of thin masonry vaults with plaster at the intrados and extrados, separated one from another with brick arches of 1 head of thickness ('una testa') and tie rods. On top of the arches, probably in 1954, two arches with a function similar to those of 'frenelli' were built. Two lateral chapels have the entrance from the central span. The roof structureof the principal nave was made of the timber trusses, while the rest of the church is made of timber beams. (A detailed report is avaailable on the dimensions of each timber element-p.10 relazione geometrico strutturale).	Bricks and lime mortar (variable thickness) for the walls.		Neogothic church built in 1906 and inaugurated in 1909. Made with the bricks from the an old church located in Golena di Po. In the 50's the apse and the choir were rebuilt in neogothic style, built before with a semicilindrical wall covered with a vault 1/4 of sphere. Pilasters and chapters before only covered with plaster were covered with marble. The vault of the principal nave has lunettes in correspondance of the two lateral chapels with ribs. In the 1996, restaruration works of the bell tower, built in 1953-1954 with a reinforced concrete structure. The walls between the pillars and beams of the structure are made of bricks. In 1929, a pinnacle in the façade was rebuilt and the groin vaults and roof consolidated. In 1946, the apse was rebuilt with a hexagonal shape instead of semicircular, and the central window was substituted with two lateral openings always in the apse walls. In 1948, a transept was built in the right with material of old buildings. Realization of the vault made of perforated bricks with ogival shape and ribs. Realization of a new roof with purlins and beams and a new entrance door. After the storm of the 1951, the church was severly damaged. In this occasion the foundation were enlarged and some tie rods were inserted at the roof level with probably some 'frenelli'(indeed they have a different construction phase from the origins of the church. In 1984, the roof was restored and in 2003, some interventions to avoid the water infiltration were executed. In 1981, the façade was restored	The cracks were visible in the arches and continuing in the vaults in the intrados. They have a variable dimension from 5 to 15 mm and in some cases cross the section of the vault. Detachment of the groin vaults from the vertical msonry structures caused by the lack of connection. The interior plaster which covers the vaults made is detached from the masonry walls.	**Description of suggested structural works (A22.1 form of damage)** Consolidation of thin brick layer vaults in all the church. Injection of cracks in the walls. Check of the existing tie rods. The roof level was not ispected because of the small space between the extrados of the vaults and the roof.		**Interventions in the thick brick layer vaults with a new system of 'frenelli'.** In the extrados of the vaults, at the impost of the vaults the rubble material used as infill was substituted with expanded clay. The vaults were retrofitted with steel chocks from the intrados and a fiber glass net with mortar in the extrados to restore the structural continuity of the vaults. The 'frenelli' were realised at the extrados of the vaults with hollow brick (12x24x24). **Intervention in the tie rods.** Some of the existing tie rods where no more efficient or inapropriatelly realised. They were cut and left onsite as historical proof and two new tie rods were built and visible from the interior of the nave at 1/3 of the height. The anchorage was put behind the raiin water collector not to have a big impact on the elevations. **In-plane steel diaphragm between the extrados of the vaults and the roof** The steel structure connects the masonry walls with the chord of the trusses and provides in plane stiffness thanks to the diagonal steel elements that connects one truss with the other. (A detailed description of the elements is in p.10 Relazione FL as built). **Reinforcement of the principal and secondary beams of the** Brackets are inserted to connect purlins, the principal central beam of the roof ('trave di colmo') and each truss. Similar brackets were used to connect the different components of the trusses. Two strips of wooden pannels were used per each side of the roof (one close to the connection between the truss and the masonry wall and one close to the top of the roof).
Castellucchio - Loc. Sarginesco	Via Mainolda 14	Sant'Andrea Apostolo	1757-1786	Urban centre	The façade with doric orden ('ordine dorico') The church has a jubé on top of the entrance The church has a single central rectangular nave with five spans, a presbitery, and a sacresty. The vaults are alternatively covered with barrel vaults and barrel vaults with lunettes.The vaults are separated with arches (3 heads='a tre teste') with tie rods. The same is for the presbitery and the apse area. From the exterior, solid buttresses were built in correspondance of the arches. The vaults are made of thin brick layer with ribs with two heads ('a due teste'). The structure of the roof is made of: longitudinal trusses supported by small pillars on top the principal arches, secodnary beams, timber planks (2,5 cm). The façade has a lofty gable. Next to the presbitery, there are two annexed bodies, a chapel (covered with a thin brick layer vault it is a cloister vault) and an ausiliar room. Next to the church on the side of the			1883 restauration of the façade In the 90's, the roof was repaired.	Overturning of the façade that caused cracks in the longitudinal walls. The most damaged system arc-vault was close to the façade. Cracks in the longitudinal walls in correspondance of the arches and openings. Cracks in the triumphal arch Overturning of the apse. Some cracks were already present before the earthquake and worsen with it. The ribs in the vaults save the vaults from worsen damage. No significant damage was observed in the system arc-pillar with the tie rods had a good response.	**Description of suggested structural works (A22.1 form of damage)** Infill of the crack, check of the existing tie rods in the arches in the principal nave and in the presbitery. Anchorage of the lofty elements.		Longitudinal tie rods were inserted at the cornice level Transversaltie rods were inserted in the back of the façade as well as in the apse and presbitery area. The anchorage is not visible and is covered with plaster. Elicoidal bars were inserted in the triumphal arc at the key and in the arches of the nave where they have wider dimensions. When the dimension was not coniderable, the cracks were infilled with timber or iron chocks and grout injection. The same was done for the crakcs in thevaults.

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Municipality	Address	Name of the church	Year	Position	Description of the church	Construction Materials	Construction Tecnhiques	Historical information	Damage post 2012 earthquake	Interventions	Post-earthquake safety intervention report	Interventions post-2012 earthquake
					ausilian room there is the bell tower with an indipendent structure.							
Felonica	Via Garibaldi 1	Assunzione della Beata Vergine Maria	1075 rebuilt	Connected on 1 side Urban centre	The church has a single nave with a square apse in the back with a semicircular choir. In the narthex, the bell tower occupies the left side and becames a characterising element of the façade.			In XV century, the church was decorated to obtain the present aspect with romanic and gothic features. The church was subjected to numerous intervention and restauration works	Not accessible the area of the roof of the apse Cracks (not big dimensions) in the façade in correspondance of the connection between the bell tower and rest of the façade. Cracks in the vertical walls and vault in the presbitery. **Additional structural problemspointed out during the intervention works** Irregular geometry of the longitudinal walls not respecting the vertical and showing an evident curvature towards the exterior, in particular of the south side. This problems were generated by the realization of the church itself, indeed already at the base, the walls are not straight. Any rotation or expulsion of the roof beams was observed in order to attribute such effect to the overturning mechanism of te walls. Such geometry condition determined that the longitudinal tie rod at the interior of the church can not be inserted because could compromise the stabolty of the walls. Terracotta tiles of the roof were subjected to sliding in some part of the roof requiring an intervention. Cracks in the pinnacles of the façade at 1/3 of its height putting at risk its stbility (possible overturning of the upper portion of the pinnacle).	**Description of suggested structural works (A22.1 form of damage)** Stiffening diaphragm in the roof Tie rods in the nartex/bell tower area Infill of cracks Check of the support of the roof beams Consolidation and seismic improvement of the sacresty Consolidation of the pinnacle in the façade		**Proposed at the first project** A transversal tie rod at 4,10/4,80m and another at 8,45m. Longitudinal tie rod at 8,05m at the interior of the curch Consolidation of the masonry of the bell tower and of the entrance area of the church with injections in the crakcs. Consolidation with reinforced fiber grout in the extrados of the vaults of the rooms on a side of the entrance. **Intervention after pointing out additional structural problems** Insertion of triangular strip wood on top of the secondary beam of the roof and of timber brackets to avoid the sliding of the terrcotta tiles and steel tape on top. In this occasion, a longitudinal tie rod at the eave level was inserted instead of the tie rod at the interior of the church at a lower height. The longitudinal tie rod has a steel plate connected with the tie rod of the bell tower/façade. Steel strips will be used to consolidate the pinnacle. Any consolidation in the extrados of the vautls close to the façade are realised but only infill cracks from the intrados. The tie rod at the 4,10 m is shifted to 4,80m and is substituted by two parallel tie rods to the masonry walls.
Roncoferraro - Loc. Villa Garibaldi	Via rodoni 2	Santi Giacomo e Mariano Martiri	1711-1730/1753	Connected on 1 side Urban centre	The façade looks at south-east and it is subdivided in three parts by four pilasters: one central with a principal door in the middle of the façade and a central rectangular window on top of the entrance; and the two lateral parts characterised by three niches one on top the other. A cornice separate the rest of the façade from the gable with a central rose window. A step back the plane of the façade on the right the wall of the lateral chapels appear At the left, the bell tower appears with an octagonal belfry and a spire. The central nave has a rectangular shape with arches that give access to the lateral chapels. It is covered with a reed vault. The presbitery with a rectangular shape is covered with a vault made of bricks	bricks for the walls the vault of the principal nave is made of reeds		1897 roof repairing 1931 restauration works that interested the repairing of the bell tower and of the plaster.	On the left side, in correspondance of the walls that structurally connect the bell tower with the other vertical walls of the church some vertical cracks appear caused by the pounding effect. The same type of cracks appears on the other side of the bell tower in correspondance of the walls of connection with the rest of the church, in particular with a lateral chapel. The cracks develop vertically and continue in the vault. In the arches of the central nave that give access to the chapels on the right some cracks, due to the seismic shaking, were found in the connection of the walls. In the arches of connection of the lateral chapels of the right some cracks of detachment were visible. The reed vault of the central nave show a crack of detachment of the vault from the façade Considerations on the damage The damage is due both the seismic event and to some existing structural problems such as: The connection of two indipendent structures (the bell tower and the rest of the church) causing a pouching effect during the earthquake. The asymmetry of the structure between the two sides of the church (left and right lateral chapels) as vertical development. The good dissipative capacity of the reed vault and roof in comparison with the supporting vertical structure that caused the detachment of the reed vault from the façade wall (maximum detachment of 3 cm).	**Description of suggested structural works (A22.1 form of damage)** Infill of the cracks in arches, vaults, and masonry walls Restore of the structural continuity between the reed vault and the masonry of the nave Repairing of a portion of the roof between the bell tower and the central nave Check of the efficiency of the existing tie rods Insertion of new tie rods It is sugested to check the possibility to isolate the structural indipendency of the bell tower.		Repairing of the exterior masonry walls of connection between the bell tower and the façade. This is realised with an operation of 'cuci-scuci' to interlock these two structures built in different moment in order to avoid other pounching effects even if a certain deformtion capacity is mantained considering the different period of vibration bell towwer and church. Repairing and seismic retrofitting of the lateral chapels (left and right corner). Insertion of tie rods attached ('a ridosso') to the transversal walls of the chapels at the level of the extrados of the vaults. The tie rods have stell plate in correspondance of the external walls and of the pillars of the nave. Restoring of the arches in the transversal rooms of connection between the chapels: consolidation of the vaults of the chapels Insertion of two pairs of tie rods in the façade in correspondance of the longitudinal walls of the central nave. Reconnection of the cracks between the vault and the façade with jute fibers and plaster in the intrados to restore the continuity.
Serravalle a Po - Loc. Libiola	Via Italia 34	Santa Cecilia Vergine e Martire	1640-1647 (new church in a new area of Libiola)	Connected with 2 buildings built as aggregate of the urban centre.	The church is composed of three naves with three entrance doors The principal nave is covered with a barrel vault with some stiffening aort of arches while the lateral naves with groin vaults the roof.A semidome covers the apse. At 30° a tie rod is inserted in ech arch and at the middle of each portion of the barrel vault of the principal nave. The curch is oriented according the axis sourth/north and the bell tower is on the west side with a rectangular plan connected to the church structure and to the presbitery area. The 'Frenelli' up to 30° of the arches of the barrel vault in the principal nave were found inspecting the area under the roof.			In the 1700, a group of builders used to work in the area of Po stimulating the spread ofsome characters typical of the baroque architectrue with the excesses of the manierism and characterised by pilasters that give rythem to the church space with cornices and a system of vaults. The bell tower was built in continuity with the presbitery and it has at the top a cuspid as in the late medievaal era. This original state was caanged in 1933-1934 when it was raised with brick texture and decoration to recall a medieval tower.	Cracks in the lateral walls at the connection of the façade showing an activation of the overturning mechanim. Vertical cracks in the lateral walls for in-plane mechanismsin portions where the masonry texture is weak. Horizontal cracks that cross the wall section at the point of connection between the roof of the lateral nave and the external longitudinal walls of the principal nave. From the survey at the interior of the roof, it is clear that the roof of the lateral naves was rebuilt higher that the precendent one because the portion of the walls of the principal nave is covered with plaster as before it was an external wall. somecrack at the connection with the aggregate as pouching effect. Some small cracks in correspondance of the corners ore due to the type of material and mortar of poor quality. Some cracks cross the section of the arches and the cornices of the lateral walls other principal nave. Cracks in the vaults, in particular in the portion close to the façade in consequence of the overturning of the façade and cross cracks in the plane of the vault with collapse of portion of plaster and sometimes bricks. (crossing the section of the vault) Cracks crossing the section of the groin vaults of the lateral naves and close to the triumphal arch between the choir and the central nave. some rafters that transfer the loads on te arches of the lateral naves that separate one vault from the other. The principal nave has timber trusses as roof structure. Some secondary beams where found broken in consequence of their small section. The roof with only one inclination of the presbitery is made of timber beams called "varese". This portion of the roof was built in later times. The bell tower has sligth cracks in some walls, not significant damage.	Inaccessibility of the roof **Description of suggested structural works (A22.1 form of damage)** Evaluate if it is the case of adding a tie rod to the triumphal arc. In the 30'-40' the first two spans of the vault close to the triumpal arc were retroffitted with the application of a metallic net and a layer of cement in the extrados. To be monitored the vertical cracks existingin the bell tower.		Injection of hydraulic mortar in the cracks of the vaults. Action of 'cuci-scuci' in the cracks of the lateral walls of the principal nave (from the exterior) and insertion in some cases of steel chocks. The same was performed in the interior walls of the area under the roof in the lateral naves. Insertion of two longitudinal tie rods at the level of the cornice at the interior of the church. The anchorage plates are visible in the façade. The tie rods also cross the wall of the bell tower and of the room annexed to the presbitery. A system against the overtrning of the façade is realised with metal beams (UPN200) connected with tie rods connected with the masonry with reinforced stiching. Consolidation of the vaults with hydraulic mortar and metal chocks and through the application in the extrados of a fiber reinforced net. A collaborative cup is executed on top of the vaults of the principal nave removing the infill material present up to the 30° in the vaults. The roof of the lateral naves is reinforced the connection between the beam elements of the roof with insertion of steel screws. The rafters on top of the arches have been substituted with inclined beams inserted in the walls. To reduce the trhust effect of the roof some tie rods to connect the walls are inserted. The beams with reduced section are substituted the terracotta tiles substituted with timber planks.

APPENDIX III

TSK-FORM OF SAN BARTOLOMEO CHURCH IN QUISTELLO (MN)

TSK-FORM

GENERAL SECTION

Identification number TSK-Form: ____________

Components of the group of compilers ____________

SEISMIC VULNERABILITY AWARENESS

D. LEVEL

- High
- Medium
- Low

A. PREVENTIVE DAMAGE CULTURE IN THE CONSTRUCTION PRACTICE

None | Moderate | High

B. PRESENCE OF ANTI-SEISMIC DAMAGE DETAILS IN THE HISTORICAL BUILDING STOCK

None | Moderate | High

B1. HISTORICAL AWARENESS (presence of anti-seismic details that belong to the rule of art)

B2. MODERN AWARENESS (interventions of retrofitting with modern techniques and materials)

C. NOTES

RECENT SEISMIC EVENTS IN THE ADMINISTRATIVE REGION (MI*≥ V)

* Micro-seismic Intensity

NOTES

15/03/1988 12:03 Reggiano earthquake (MI=5, 4.57 Mw) ____________

- In the last 100 years
- In the last 50 years
- In the last 20 years
- In the last 10 years
- Other

COMMON TYPOLOGY OF THE AREA

Plan | Façade | Period

NOTES

Baroque typology (3 naves and façade that reflects the tripartition of naves and a soaring gable)

LEGEND FOR THE DETAIL SECTION

A. TERRITORIAL MACRO-ELEMENTS

B. TERRITORIAL VULENRABILITY MODIFIER

They change according to the territory

E. MECHANISMS

The correspondence with the existing methodology

C.= Level of vulnerability for the territorial vulnerability modifier

D.= Quality of the assessment

C. & D. level: High (3 dots), Medium (2 dots), Low (1 dot)

F. = Level of vulnerability for the assessed macro-element

F. level: High (5 dots), Medium-High (4 dots), Medium (3 dots), Medium-Low (2 dots), Low (1 dot)

DETAIL SECTION

IDENTIFICATION OF THE CHURCH

Name of the church: _San Bartolomeo_________ Address: __Via Cesare Battisti 22_______________Coordinates: ________45.008373, 10.980835____________________

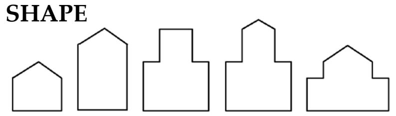
TERRITORIAL MACRO-ELEMENT	TERRITORIAL VULNERABILITY MODIFIER	MECHANISMS

FAÇADE

F. ● ● ● ● ○

CONFIGURATION

SHAPE



○ ○ ○ ○ ●

Attached sketch with size

Is it slender? Yes No

C ○ ○ ○
D ● ● ●

PRESENCE OF OPENINGS Yes No

C ● ○ ○
D ● ● ●

Are they many in proportion to the dimension of the façade? Yes No

Position: 1st order | Close one to the other | 2nd order | Close to the corner | In the gable | In the middle of the 2nd order

PRESENCE OF A ROSE WINDOW Yes No

C ○ ○ ○
D ● ● ●

Existing in the past, now closed

Shape: round | Other

Position: Close to the cornice | Close to the border | In the middle of the façade

SHAPE OF THE GABLE Triangular | 'All'Abruzzese'

C ● ● ●
D ● ● ●

- Is it soaring? Yes No
- Presence of cornice at the base of the gable Yes No
- Presence of anti-overturning details Yes No Specify________________________

PRESENCE OF CORNICES Yes No

C ● ● ●
D ● ● ●

Position: Between the 1st and 2nd order | To separate the gable

PRESENCE OF NICHES Yes No

C ● ● ●
D ● ● ●

Are they symmetric? Yes No

STATE OF CONSERVATION

PRESENCE OF CRACKS Yes No

C ○ ○ ○
D ● ● ●

Specify_________________________

PRESENCE OF INTERVENTIONS IN THE MASONRY Yes No

C ○ ○ ○
D ● ● ●

Specify_________________________

BOUNDARY CONDITIONS

ANCHORAGE AT THE TOP OF THE FAÇADE Yes No

C ● ● ●
D ● ● ●

- ○ Presence of concrete ring beam
- ○ Longitudinal roof beam (potential pounding effect)
- ○ Presence of anti-overturning metal system
- ○ Presence of a roof system that guarantees a box-like behaviour

CONNECTION WITH THE LONGITUDINAL WALLS

C ○ ○ ○
D ● ● ●

└ ● ┴ ○

Interlocking guaranteed by the brick construction technique

MASONRY

Multi-leaf | No identifiable leaves

MATERIAL

Brick | Stone | Mixed: bricks & stones

Fabric & quality of the masonry

Walls made of bricks with staggered vertical mortar joints and horizontal bed joints

MORTAR

Thickness __About 2 cm thick____________

Composition _______N/A_______________

State of conservation _____Good______

ANTI-OVERTURNING ELEMENTS

PRESENCE OF PILASTERS Yes No

C ○ ○ ○
D ● ● ●

Same material of the façade? Yes No Specify_____________________

Are they in correspondence with the nave distribution? Yes No Specify_____________________

PRESENCE OF TIE RODS Yes No

C ● ● ●
D ● ● ●

Efficiency? Yes No

Tie rod restrains? Visible | Inserted in the masonry

Specify what type_____________________

MECHANISMS

IN PLANE:

- Mechanism 3;
- Mechanism 6;
- Mechanism 11;
- Mechanism 17;
- Mechanism 23

OUT-OF-PLANE:

- Mechanism 1;
- Mechanism 2;
- Mechanism 5;
- Mechanism 10;
- Mechanism 16;
- Mechanism 22

OTHER WALLS

F. ● ● ● ○ ○

* Specify the assessed wall in case the conditions differ one from the other

CONFIGURATION

All the walls presents similar characteristics

Position: 1st order | 2nd order

Are they slender? Yes No

C ○ ○ ○
D ● ● ●

PRESENCE OF OPENINGS Yes No

C ● ○ ○
D ● ● ●

Are they many in proportion to the dimension of the assessed wall? Yes No

Position: 1st order | Close one to the other | 2nd order | Close to the corner

MASONRY

Multi-leaf | No identifiable leaves

MATERIAL

Brick | Stone | Mixed: bricks & stones

Fabric & quality of the masonry

Walls made of bricks with staggered vertical mortar joints and horizontal bed joints

MORTAR

Thickness ___About 2 cm thick_____

Composition ________N/A__________

State of conservation ______Good_____

BOUNDARY CONDITIONS

ANCHORAGE AT THE TOP Yes No

C ● ● ●
D ● ● ●

- ○ Presence of concrete ring beam
- ● Roof beam (potential pounding effect)
- ○ Presence of anti-overturning metal system
- ○ Presence of a roof system that guarantees a box-like behaviour

CONNECTION WITH THE OTHER WALLS

C ○ ○ ○
D ● ● ●

└ ● ┴ ○

Specify_____________

MECHANISMS

- Mechanism 19;
- Mechanism 20;
- Mechanism 21

STATE OF CONSERVATION

PRESENCE OF CRACKS — C ○ ○ ○ / D ● ● ●
Yes | No

PRESENCE OF INTERVENTIONS IN THE MASONRY — C ○ ○ ○ / D ● ● ●
Yes | No

ANTI-OVERTURNING ELEMENTS

PRESENCE OF PILASTERS — C ○ ○ ○ / D ● ● ●
Yes | No
Same material of the façade?
Yes | No | Specify________________
Are they in correspondence with the nave distribution?
Yes | No | Specify________________

PRESENCE OF TIE RODS — C ● ● ● / D ● ● ●
Yes | No
Efficiency?
Yes | No | Not- known
Tie rod restrains?
Visible | Inserted in the masonry
Specify what type________________

BELL TOWER

F. ○ ○ ○ ○ ○

CONFIGURATION

○ Isolated
○ Annexed as part of the church structure
○ Annexed, but structurally independent

Slender — C ○ ○ ○ / D ○ ○ ○
Yes | No
Plan Size ____________
Thickness of the walls________________

PRESENCE OF BELFRY — C ○ ○ ○ / D ○ ○ ○
Yes | No
Are the pillars of the belfry slender? Yes | No
Same material as the rest of the bell tower structure?
Yes | No | Specify____________

PRESENCE OF OPENINGS Yes | No Size____________ — C ○ ○ ○ / D ○ ○ ○
How many? Many | Few | None
Position ____________ Close one to the other | Close to the corner

MASONRY

Multi-leaf | No identifiable leaves

MATERIAL
Brick | Stone | Mixed: bricks & stones

Fabric & quality of the masonry
Refer to (Borri et al. 2015)
Specify________________

MORTAR
Thickness ________________
Composition ________________
State of conservation ________________

ANTI-SEISMIC DETAILS

PRESENCE OF PILASTERS — C ○ ○ ○ / D ○ ○ ○
Yes | No
Same material of the bell tower?
Yes | No | Specify________________

PRESENCE OF TIE RODS — C ○ ○ ○ / D ○ ○ ○
Yes | No
Efficiency? Yes | No
Tie rod restrains? Visible | Inserted in the masonry

BOUNDARY CONDITIONS

CONNECTION WITH THE CHURCH STRUCTURE — C ○ ○ ○ / D ○ ○ ○
○ Attached with to the triumphal arch
○ Attached to the rest of the church structure

STATE OF CONSERVATION

PRESENCE OF CRACKS — C ○ ○ ○ / D ○ ○ ○
Yes | No
Specify________________
○ Attached photographic documentation

PRESENCE OF INTERVENTIONS IN THE MASONRY — C ○ ○ ○ / D ○ ○ ○
Yes | No
Specify________________
○ Attached photographic documentation

- Mechanism 27;
- Mechanism 28

COLONNADE

F. ● ○ ○ ○ ○

CONFIGURATION

C ● ○ ○ / D ○ ○ ○
Is it formed as a series of stone columns & arches?
Yes | No
Is it built as pilasters and arches?
Yes | No
Is there an entablature?
Yes | No

MASONRY

MATERIAL
Brick | Stone | Mixed: bricks & stones

Fabric & quality of the masonry
Walls made of bricks with staggered vertical mortar joints and horizontal bed joints

MORTAR Inspectable? Yes | No
Thickness ________________
Composition ________________
State of conservation ________________

STATE OF CONSERVATION

PRESENCE OF CRACKS — C ○ ○ ○ / D ● ● ●
Yes | No

PRESENCE OF INTERVENTIONS IN THE MASONRY — C ○ ○ ○ / D ● ● ●
Yes | No

- Mechanism 7

ARCHES

F. ● ○ ○ ○ ○

* Specify the assessed arch in case the conditions differ one from the other

CONFIGURATION

Is the arch of the apse confined by walls?
Yes | No
Do they form a structural grid?
Yes | No

Is there a triumphal arch? — C ● ○ ○ / D ● ● ●
Yes | No
Are they fabricated in order to allow the intersection of the adjacent vaults?
Yes | No

MASONRY

MATERIAL
Brick | Stone
Two wythes? Yes | No
MORTAR Not inspectable
Good state of conservation

STATE OF CONSERVATION

PRESENCE OF CRACKS Yes | No — C ○ ○ ○ / D ● ● ●

PRESENCE OF INTERVENTIONS IN THE MASONRY Yes | No — C ○ ○ ○ / D ● ● ●
Specify________________

VAULTS

F. ● ● ● ● ○

* Specify the assessed vault in case the conditions differ one from the other

* Please attach the soffit plan

CONFIGURATION

C ● ● ○ / D ● ● ●

- ○ Barrel vault
- ○ Groin vault
- ● Vault with lunettes
- ○ Other. Specify_______________

- Are they consecutive? Yes / No
- Are they interrupted by arches? **Yes** / No
- Is there a vault adjacent to the façade? **Yes** / No

PRESENCE OF RIBS C ● ● ● / D ● ● ●

Yes / **No**

Position of the ribs: Intrados / Extrados

Specify in what vault_______________

ANTI-SEISMIC DETAILS

PRESENCE OF TIE RODS C ● ● ● / D ● ● ●

Yes / **No**

Specify the position _______________

Specify what vault _______________

BOUNDARY CONDITIONS

C ● ● ● / D ● ● ●

- Is the first vault of the nave in direct contact with the façade? **Yes** / No
- Is there an arch adjacent to the façade? Yes / **No**
- Are there any soaring elements in proximity of the vaults that could cause damage by collapsing? **Yes** / No

MASONRY

MATERIAL C ● ● ● / D ● ● ●

Brick / Stone / Timber and reeds

FABRIC

Vertical stacked bond / Horizontal stacked bond

''voissoir'' vault / 'volte in folio'

MORTAR Inspectable? Yes / **No**

Thickness _______________

Composition _______________

State of conservation _______________

STATE OF CONSERVATION

PRESENCE OF CRACKS C ○ ○ ○ / D ● ● ●

Yes / **No**

Specify_______________

○ Attached photographic documentation

PRESENCE OF INTERVENTIONS IN THE MASONRY C ○ ○ ○ / D ● ● ●

Yes / **No**

Specify_______________

○ Attached photographic documentation

- Mechanism 8;
- Mechanism 9;
- Mechanism 18;
- Mechanism 12;
- Mechanism 24

DOME

F. ● ● ○ ○ ○

CONFIGURATION

Is it part of a structurally independent chapel? Yes / **No** C ○ ○ ○ / D ● ● ●

Specify the position with respect to the church structural layout At the centre of the plan configuration

PRESENCE OF RIBS C ● ● ● / D ● ● ●

Yes / **No**

Position of the ribs: Intrados / Extrados

Are they structurally independent from the dome? (useful in some cases in the construction phase as centring) Yes / No

Are they decorative? Yes / No

Are they annexed or integrated to the dome? Yes / No

BOUNDARY CONDITIONS

C ● ○ ○ / D ● ● ●

How is the dome supported?

- ○ by walls
- ○ by pillars
- ○ by a tambour
- ○ by recessed niches
- ○ by squinch projecting cornices
- ○ by arch shaped niches
- ● by arches connected with underneath vaults

Other. Specify_______________

STATE OF CONSERVATION

PRESENCE OF CRACKS C ○ ○ ○ / D ● ● ●

Yes / **No**

Specify_______________

PRESENCE OF INTERVENTIONS? C ○ ○ ○ / D ● ● ●

Yes / **No**

Specify_______________

MASONRY

MATERIAL C ● ○ ○ / D ● ● ●

Brick / Stone / **Timber and reeds**

FABRIC

Vertical stacked bond / Horizontal stacked bond

''voissoir'' dome

MORTAR Inspectable? Yes / No

Join thickness _______________

Composition _______________

State of conservation _______________

ANTI-SEISMIC DETAILS

METAL RETAINING ELEMENTS C ● ● ● / D ● ● ●

Yes / **No**

Specify the position _______________

OTHER RETROFITTING SYSTEMS

Yes / No

Specify _______________

- Mechanism 14;
- Mechanism 15

FAÇADE
ADDITIONAL INFO

OTHER WALLS
ADDITIONAL INFO

BELL TOWER
ADDITIONAL INFO

COLONNADE
ADDITIONAL INFO

ARCHES
ADDITIONAL INFO

VAULTS
ADDITIONAL INFO

DOME
ADDITIONAL INFO

APPENDIX
IV

INFORMATION ON THE VISITED CHURCHES IN MONTREAL, QUÉBEC

Identification of the church				Materials & Structure						Information on interventions and changes		Other information
Municipality	Address	Name of the church	Year	Canadian heritage classification	Façade configuration	Plan configuration	Bell tower	Perimetral wall material	Roof material	Interventions	Changes	Other infos
Montréal	3435, Chemin de la Côte-Sainte-Catherine H3T 1C7	Côte-des-Neiges Presbyterian	1888-1892	Medium-high (C)	1 - Conesfroy	Rectangular	A step back from the façade plane, in the middle of the façade	Cut stone	Metal covering	Addition of the porch and the semicircular arch of the façade in a later moment	/	
Montréal	1855, rue Rachel Est - H2H 1P5	Immaculée-Conception	1895-1898	Medium-high (C)	3 - Néo-roman	Latin cross, overhanging choir, semicircular apse	Bell tower in the middle of the façade	Roughly stone at the base and speckled stone buttress	Slate	2008: Repairing works of the bell tower 2008: Reparing of the joints of mortar 2009: Repairing works and substitution of the roof covering	/	Similarity between the façades of this church and Sainte-Brigide-de-Kildare
Montréal	1847, boulevard Gouin - H2C 1C8	La Visitation de la Bienheureuse-Vierge-Marie	1749-1751	High (A)	2 - Baillargé	Rectangular, overhanging choir	Two bell towers enclosing the hall space.	Stones of a thickness of 3 ft	Slate	1964: Replacement of buttresses with new concrete ones 1752: Addition of buttresses 1814: Repairing works in the roof 1850: Addition of a new façade made of limestones	Several restoration works of the interior of the building. 1761 to 1772: Construction of the Sacristy 1844: Enlargement of the Sacristy	1749: Beginning of the construction 1751: End of construction 1850: Enlargement - The nave is extended with the construction of a new facade
Montréal	3200, rue Ontario - H1V 2S1	Nativité-de-la-Sainte-Vierge	1921-1925	Medium-high (C)	/	overhanging choir, semicircular apse	Imposing bell tower	stones	Copper	1922: Construction of the church on the ruins of an old church that was destroyed by a fire	/	Vault covered with clay tiles
Montréal	400, rue Saint-Paul Est - H2Y 1H4	Église dite chapelle Notre-Dame-de-Bon-Secours	1771-1773	High (A)	2 - Baillargé	Rectangular, overhanging choir, apse polygonal shape	1 central bell tower, 2 side steeples 1 octagonal tower is built on the apse	stones	Metal covering	Demolition of buildings annexed to the apse, substitution of some stones in the curtain walls, discovery of archaeological remains, valorization of the crypt	Addition of a new façade over the old one in 1892 with a central bell tower replaced in 1952 due to structural failure of another very close to the first one of 1678	1754: The chapel was destroyed by fire 1771: Reconstruction is undertaken 1885-1886: Modification of the bell tower and the façade From the visit (25-09-2018), the church seems in good conditions.
Montréal	5333 Avenue Notre-Dame-de-Grâce - H4A 1L2	Église Notre-Dame-de-Grâce	1851-1853	Medium-high (C)	Italian barroque	Latin cross, projecting heart, apse in hemicycle	/	stones	Copper	Addition of a bell tower at the end of the transept in 1923 Internal modifications in 1925 then 1965 1959: Repairing works of the roof 2000: Repairing works of the façade	1963: Enlargement of the sacristy	1851: Beginning of the original construction 1853: End of original works 1926: Construction of the towers
Montréal	5959 Boulevard Monk, Montréal - H4E 3H5	Notre-Dame-du-Perpétuel-Secours	1914-1920	Medium (D)	2 - Baillargé	Latin cross choir overhanging right apse	Two bell towers enclosing the hall space. They are asymmetrical	stones	asphalt	/	/	1657: Original timber construction 1675: Modification to realize it recurring to masonry 1754: Complete destruction (except foundation) caused by fire 1777 to 1783: New church without trasnsept on the old foundations Wooden vault hanging from the frame
Montréal	665, rue de l'Église - H9S 1R4	Présentation-de-la-Sainte-Vierge	1900-1901	Medium (D)	1 - Conesfroy 3 - Néo-roman	Rectangular, overhanging choir, apse in semi-circle	A step back from the façade plane, in the middle of the façade	limestone	sheet of metal	1939: Repairing works of the roof 2016: Repairing works of the roof	Addition of the covered access	1900: Beginning of original construction 1901: End of original works From the visit (05-09-2018), the church seems in good conditions.
Montréal	454 Avenue Laurier E, Montréal - H2J 1E7	Saint-Denis	1911-1913 1900-1945	Medium-low (E)	2 - Baillargé	Greek cross	Two bell towers enclosing the hall space. They are asymmetrical	stones	Copper	Largely destroyed by a fire in January 1931 and rebuilt the same year following the same plan	/	
Montréal	2851, rue Masson - H1Y 1X1	Saint-Esprit-de-Rosemont	1931-1933	Medium-high (C)	3 - Néo-roman	Latin cross, overhanging choir, semicircular apse	In the middle of the façade	stones	Copper	1984: Repairing of the roof 1997: Repairing of the masonry 2000: Installation of tie rods 2003: Roof repairing and masonry interventions	/	1931: Begining of the original construction 1933: End of original works
Montréal	*10050, boulevard Gouin Est - H1C 1A8*	Saint-Joseph	1875-1876	Medium-high (C)	3 - Néo-roman	Rectangular, projecting choir, polygonal apse	In the middle of the façade	rubble limestone	Metal covering	2000: Repairing of the floor 1937: Bell tower destroyed by lightning, rebuilt identically	/	Estimate of the sacristy and the extension of the choir available
Montréal	5525, rue Jarry Est - H1P 1V1	Saint-Léonard	1907-1908	Medium-low (E)	3 - Néo-roman	Latin cross, overhanging choir, semicircular apse	In the middle of the façade	stones	Metal covering	1907: Rebuilt completed after the total destruction by a fire 2000: Repairing of the roof	1930: After a fire the perimetral walls were safe and the interior as well as the bell tower were rebuilt	1907: Begining of the original construction 1908: End of the original construction
Montréal	2602, rue Beaubien Est - H1Y 1G5	Saint-Marc	1931-1932	Medium (D)	2 - Baillargé	Latin cross, overhanging choir, apse	Two bell towers enclosing the hall space	stones	Copper	1966: Renovation of the sanctuary 1974: Repairing of the roof 1988: Completely renovated forecourt	1962: Substitution of the wooden steps with concrete steps	Transformed in a community center 1931: Beginning of original construction 1932: End of original work
Montréal	1174, rue De Champlain - H2L 2R8	Sainte-Brigide-de-Kildare	1878-1880	Medium-high (C)	3 - Néo-roman	Rectangular, overhanging choir	In the middle of the façade. The most beautiful bell tower in montreal according to Gérard morisset	limestone	Asphalt	Construction of the bell tower between 1885 and 1886	Structural problems that make the renovation works difficult	Big problems which may lead to its demolition despite the conservation of its facade 1878: Construction of the church
Montréal	16037, boulevard Gouin Ouest - H9H 1C7	Sainte-Geneviève	1843-1844	High (A)	2 - Baillargé	Rectangular, overhanging choir	Two bell towers enclosing the hall space	limestone	Metal covering	Construction over an old church Addition of 2 concrete columns to support the jubé 1925: Repair of the facade	Bell towers replaced in 1909, repairing works of the façade, interior modifications over the years	Unique work of baillargé in montréal 1843: Beginning of the original construction 1844: End of the original construction 1909: Repairing of arrows

APPENDIX V

TSK-FORM OF ST. JOSEPH IN MONTRÉAL

TSK-FORM FOR THE NÉO-ROMAN TYPOLOGY

GENERAL SECTION

Identification number TSK-Form: ____________

Components of the group of compilers ____________

SEISMIC VULNERABILITY AWARENESS

D. LEVEL

- High
- Medium
- Low

A. PREVENTIVE DAMAGE CULTURE IN THE CONSTRUCTION PRACTICE

Low | Medium | High

B. PRESENCE OF ANTI-SEISMIC DAMAGE DETAILS IN THE HISTORICAL BUILDING STOCK

Low | Medium | High

B1. HISTORICAL AWARENESS (presence of anti-seismic details that belong to the rule of art)

B2. MODERN AWARENESS (interventions of retrofitting with modern techniques and materials)

C. NOTES

Only a few building on the principal street of the historical centre of Montreal show some tie rods. Among churches, only Notre Dame de Bon Secour.

RECENT SEISMIC EVENTS FELT IN MONTREAL

NOTES

1988, Saguenay earthquake (5.9 *Mw*)

- In the last 100 years
- In the last 50 years
- In the last 20 years
- In the last 10 years
- Other

COMMON TYPOLOGY OF THE AREA

Plan | Façade | Period

NOTES

néo-roman typology

LEGEND FOR THE DETAIL SECTION

A. TERRITORIAL MACRO-ELEMENTS

B. TERRITORIAL VULNERABILITY MODIFIER

According to the territory

E. MECHANISMS — Mechanisms that could develop

C.= Level of vulnerability for the territorial vulnerability modifier

D.= Quality of the assessment

C. & D. level: High (3 dots), Medium (2 dots), Low (1 dot)

F. = Level of vulnerability for the assessed macro-element

F. level: High (5 dots), Medium-High (4 dots), Medium (3 dots), Medium-Low (2 dots), Low (1 dot)

DETAILED SECTION

IDENTIFICATION OF THE CHURCH

Name of the church: _St. Joseph___ **Address:** ____10050 Boulevard Gouin E, Montreal, Quebec H1C 1A8, Canada___**Coordinates:** __45.662881, - 73.561459___

TERRITORIAL MACRO-ELEMENT	TERRITORIAL VULNERABILITY MODIFIER	MECHANISMS

FAÇADE-BELL TOWER

F. ● ● ● ● ○

CONFIGURATION

SHAPE OF THE FAÇADE WALL ●

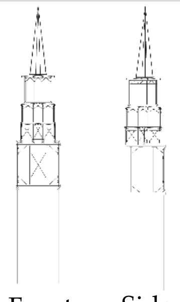
Is it slender? Yes **No**
C ● ○ ○
D ● ● ●

PRESENCE OF OPENINGS C ● ● ● / D ● ● ●
Yes No
How many? **Many** Few None
Position: 1st order / Close one to the other / **2nd order** / Close to the corner / **In the centre of the façade**

PRESENCE OF A ROSE WINDOW C ● ○ ○ / D ● ● ●
Yes No
Existing in the past, now closed
Shape: **round** Other
Position: Close to the cornice / Close to the border / **In the middle of the gable area**

SOARING GABLE C ● ● ● / D ● ● ●
Triangular **Rectangular**
- Is it soaring? **Yes** No
- Presence of a rose window or big opening at the centre of the gable **Yes** No
- Presence of anti-overturning details Yes **No** Specify___________________

STRUCTURE BELL TOWER



Slenderness of the pillars C ● ○ ○ / D ● ● ○
- 1st deck Yes **No**
- 2nd deck Yes **No**
- Top level **Not inspectable**

Robustness of the other elements **Yes** No
Average cross section 25*25 cm
C ● ○ ○ / D ● ● ○

Joints C ● ○ ○ / D ● ● ○
Tenon mortise
Are the connections closed? **Yes** No
Specify with timber pins

STATE OF CONSERVATION (timber)

PRESENCE OF HUMIDITY C ● ○ ○ / D ● ● ●
High Medium **Low**

PRESENCE OF FUNGUS C ● ● ○ / D ● ● ○
High Medium **Low**

PRESENCE OF RETROFITTING OR SUBSTITUTED ELEMENTS C ● ○ ○ / D ● ● ●
Yes **No**
Specify___________________

MATERIAL: MASONRY

Multi-leaf No identifiable leaves

MATERIAL C ● ● ○ / D ● ● ●
Stone Specify __Sandstone__

Fabric & quality of the masonry
Squared stone for the exterior leaf
Roughly squared stones for the interior leaf

MORTAR
Thickness Regular layers (about 2 cm thick)
Composition ____Lime________
State of conservation _Good_______

BOUNDARY CONDITIONS

ANCHORAGE AT THE TOP OF THE FAÇADE C ● ● ● / D ● ● ●
Yes **No**

ELEMENTS OF THE BELL TOWER ENTERING THE FAÇADE WALL C ● ● ● / D ● ● ●
Yes No
Specify At the 2nd deck for the longitudinal beams and the pillars on the façade side

CONNECTION OF THE FAÇADE WITH LONGITUDINAL WALLS C ● ○ ○ / D ● ● ●
● ○
Specify Squared block of stone to define the edge

MATERIAL: TIMBER

Softwood Hardwood C ● ● ○ / D ● ● ●
Specify __Pine______

ANTI-OVERTURNING ELEMENTS (façade wall)

PRESENCE OF BUTTRESSES C ○ ○ ○ / D ● ● ●
Yes No
Same material of the façade?
Yes No Specify_______________
Are they in correspondence with the bell tower?
Yes No Specify_______________

PRESENCE OF TIE RODS C ● ● ● / D ● ● ●
Yes **No**
Efficiency?
Yes No
Tie rod restrains?
Visible Inserted in the masonry
Specify what type_______________

STATE OF CONSERVATION

PRESENCE OF CRACKS C ● ● ● / D ● ● ●
Yes No Symmetrical
Specify Close to the openings of the second order and in the lateral windows of the longitudinal walls

PRESENCE OF INTERVENTIONS IN THE MASONRY C ● ○ ○ / D ● ● ●
Yes No
Specify In the upper part of the east longitudinal wall close to the corner with the façade

STIFFENING AND RETAINING ELEMENTS (timber structure)

PRESENCE OF BRACING ELEMENTS C ● ○ ○ / D ● ● ●
Yes No
Is their layout symmetrical?
Yes No Specify_______________
Are they part of the original structure?
Yes No Specify_______________

PRESENCE OF RETAINING METAL ELEMENTS C ● ● ● / D ● ● ●
Yes **No**

BOUNDARY CONDITIONS

ANCHORAGE TO THE MASONRY WALLS C ● ● ● / D ● ● ●
Are the supports of the timber elements well anchored to the wall structure?
Yes No Specify Timber elements are built-in in the masonry

MECHANISMS

- In plane cracks due to the high number of openings;
- Overturning of the upper part of the façade;
- A horizontal hinge in the middle of the façade may easily develop due to the presence of a large opening;
- Collapse of the top part of the bell tower structure;
- Collapse of the second deck and top part of the bell tower;
- Pounding effect of the top part of the bell tower due to the longitudinal beam of the structure entering the masonry at the second deck;

OTHER WALLS

F. ● ● ● ○ ○

* Specify the assessed wall in case the conditions differ one from the other

____

CONFIGURATION

Are they slender? C ● ○ ○ D ● ● ●

Yes | **No**

PRESENCE OF OPENINGS C ● ● ○ D ● ● ●

Yes | No

How many?

More than one per span between columns | **One per span between columns** | Less than one per span between columns

More than one per wall of the apse | One per wall of the apse | **Less than one per wall of the apse**

Position | Close one to the other | Close to the corner

MATERIAL: MASONRY

Multi-leaf | No identifiable leaves

MATERIAL C ● ● ○ D ● ● ●

Stone Specify __Sandstone__

Fabric & quality of the masonry

Roughly squared stones

MORTAR

Thickness Regular layers (about 2 cm thick)

Composition ____Lime__________

State of conservation _Good________

BOUNDARY CONDITIONS

ANCHORAGE AT THE TOP **Yes** | No C ● ● ○ D ● ● ●

- ● Presence of a retaining roof structure
- ● Roof beam (potential pounding effect) In the apse area
- ○ Presence of anti-overturning metal system
- ● Beam of the choir deck ('jubé')
- ● Annexed body: sacristy

CONNECTION WITH THE OTHER WALLS C ● ○ ○ D ● ● ●

Squared cut stones ● ○

STATE OF CONSERVATION

PRESENCE OF CRACKS C ● ● ○ D ● ● ●

Yes | No

Vertical crack (parallel to the repaired one), in the north-east longitudinal wall of the nave;
Close to the first window of both longitudinal walls

PRESENCE OF INTERVENTIONS IN THE MASONRY C ● ● ○ D ● ● ●

Yes | No

Repaired vertical crack in the north-east wall and repairing of a portion in the upper part of the wall close to the façade of the south wall

ANTI-OVERTURNING ELEMENTS

PRESENCE OF BUTTRESSES C ● ● ● D ● ● ●

Yes | **No**

Same material of the façade?

Yes | No Specify_________________

Are they in correspondence with the roof structure span?

Yes | No Specify_________________

PRESENCE OF TIE RODS C ● ● ● D ● ● ●

Yes | **No**

Efficiency?

Yes | No

Tie rod restrains?

Visible | Inserted in the masonry

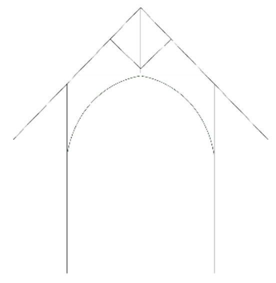
Specify what type________________

- Overturning mechanism of the walls
- In-plane mechanisms in the walls, close to the openings
- Activation of kinematic mechanisms favoured by the presence of cracks near the windows of the façade and the first window of the longitudinal walls
- Local damage due to the pounding effect on the apse from the walls of the sacristy and the roof rafters;
- Local damage due to the pounding effect on the on the longitudinal walls from the beam of the choir deck ('jubé').

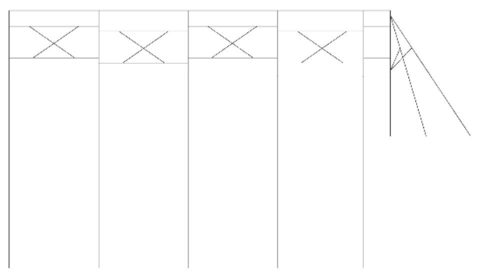
ROOF SYSTEM TRUSS - COLUMNS

F. ● ● ○ ○ ○

CONFIGURATION



Transversal direction



Longitudinal direction

Is the structural system designed just for the vertical loads? C ● ○ ○ D ● ● ○

Yes | **No** Also for horizontal actions

WHERE IS THE ROOF STRUCTURE CONNECTED TO THE MASONRY WALLS?

- ● Connected to the façade
- ● Connected to the longitudinal walls
- ● Connected to the apse wall

Robustness of other elements C ● ○ ○ D ● ● ○

Yes | No

Cross section of the main elements:

- 25*25 cm
- 20*25 cm
- 8*25 cm

Joints C ● ○ ○ D ● ● ○

Tenon mortise

Are the connections closed?

Yes | No

Specify with timber pins

MATERIAL: TIMBER

Softwood | Hardwood C ● ● ○ D ● ● ●

Specify __Pine______

STIFFENING ELEMENTS

Is there an arch that stiffens transversally the structure and reduces the free height of the pillars? C ● ○ ○ D ● ● ●

Yes | No

Are there the longitudinal beams to connect the pillars?

Yes | No

Is there a bracing structure that connects the trusses one to the other?

Yes | No

Is there a radial structure in the apse area that contrasts the longitudinal response of the entire system of trusses and longitudinal structures?

Yes | No

BOUNDARY CONDITIONS

ANCHORAGE TO THE MASONRY WALLS C ● ● ● D ● ● ●

Are the supports of the timber elements well anchored to the wall structure?

Yes | No

Specify There is a built in joint between timber elements and masonry

STATE OF CONSERVATION

PRESENCE OF HUMIDITY C ● ○ ○ D ● ● ●

High | Medium | **Low**

PRESENCE OF FUNGUS C ● ○ ○ D ● ● ○

High | Medium | **Low**

DETERIORATION OF THE HEADS OF THE TIMBER ELEMENTS AT THE SUPPORTS C ○ ○ ○ D ● ● ○

Yes | **No**

RETROFITTED OR SUBSTITUTED ELEMENTS

Yes | No C ● ○ ○ D ● ● ●

Specify tie rods and metal elements to retrofit some connections between elements of the truss

- Deterioration of the head of the timber elements causing lack of support;
- Possible disassembly of the connections with the seismic actions;
- Excessive increase of stress levels due to the seismic action compared to the capacity of the cross sections

ANNEXED BODY: THE SACRESTY WALLS

F. ● ● ○ ○ ○

CONFIGURATION

Are they slender? C ● ○ ○ / D ● ● ●
Yes | **No**

PRESENCE OF OPENINGS C ● ○ ○ / D ● ● ●
Yes | No
How many?
More than 1 | **1** | None
Position: Close one to the other | Close to the corner

MATERIAL: MASONRY

Multi-leaf | No identifiable leaves

MATERIAL C ● ● ○ / D ● ● ●
Stone Specify __Sandstone__
Fabric & quality of the masonry
Roughly squared stones
MORTAR
Thickness Regular layers (about 2 cm thick)
Composition _____Lime____________
State of conservation _Good_______

BOUNDARY CONDITIONS

ANCHORAGE AT THE TOP **Yes** | No C ● ● ○ / D ● ● ○

- ● Roof beams (potential pounding effect)
- ○ Presence of anti-overturning metal system
- ● Pounding effect of the church structure
- ○ Other

CONNECTION WITH THE OTHER WALLS C ● ○ ○ / D ● ● ●
Squared cut stones ● ○

STATE OF CONSERVATION

PRESENCE OF CRACKS C ● ● ○ / D ● ● ●
Yes | **No**
Specify_______________

PRESENCE OF INTERVENTIONS IN THE MASONRY C ● ● ○ / D ● ● ●
Yes | **No**
Specify_______________

ANTI-OVERTURNING ELEMENTS

PRESENCE OF TIE RODS C ● ● ● / D ● ● ●
Yes | **No**
Efficiency?
Yes | No
Tie rod restrains?
Visible | Inserted in the masonry
Specify what type____________________

- Overturning mechanism of the walls
- In-plane mechanisms in the walls, close to the openings
- Local damage due to the pounding effect of the roof beams

ANNEXED BODY: THE CHIMNEY

F. ● ● ● ○ ○

CONFIGURATION

Position
Annexed to the apse

Is it slender? C ● ● ● / D ● ● ●
Yes | No
C ● ● ● / D ● ● ●

MASONRY

MATERIAL C ● ○ ○ / D ● ● ●
Brick | Stone

MORTAR Not inspectable
Thin layer of mortar

STATE OF CONSERVATION

PRESENCE OF CRACKS Yes | **No** C ● ○ ○ / D ● ● ●
Specify_______________

PRESENCE OF INTERVENTIONS IN THE MASONRY C ● ○ ○ / D ● ● ●
Yes | **No**
Specify_______________

- Overturning due to the slenderness;
- Crack configuration along the mortar joints.

FAÇADE-BELL TOWER
ADDITIONAL INFO

OTHER WALLS
ADDITIONAL INFO

ROOF SYSTEM
TRUSS-COLUMNS
ADDITIONAL INFO

ANNEXED BODY:
SACRISTY WALLS
ADDITIONAL INFO

ANNEXED BODY:
THE CHIMNEY
ADDITIONAL INFO

Politecnico di Milano
Milan, April 2020