POLITECNICO DI MILANO

School of Architecture, Urban Planning and Construction Engineering

Master of Science in "*Building and Architectural Engineering*"

SEISMIC ANALYSIS OF HISTORICAL MASONRY BUILDINGS

"The Church of Assunzione della Beata Vergine Maria in Villavesco, Tavazzano"

Supervisor: Prof. Ing. Maria Adelaide Vittoria Parisi

Co-Supervisors: Ing. Marco Locatelli, Arch. Lorenzo Cantini

Master Thesis

Beatrice Formenti - 892537

Academic Year 2021/2022

Un ringramento alla Diocesi di Lodi che si è resa disponibile
per fornire il materiale necessario allo sviluppo di questa tesi.

Un ringraziamento speciale alla Prof. Ing. Maria Adelaide Vittoria Parisi,
il suo supporto e la sua determinazione sono stati fondamentali per il
raggiungimento di questo obiettivo.

ABSTRACT

The aim of this master thesis is the evaluation of the damage and structural condition of a masonry historical building, assessing its expected seismic behaviour for the level of design earthquake considered for the site in the Province of Lodi. The case study is the Church of Assunzione della Beata Vergine Maria in Villavesco, Tavazzano.

The study is carried out by initially studying the structure from the geometric point of view, proceeding with analyses that will allow to assess the composition of the structure, possible damage, and current conditions of the structure itself. Obviously, the analyses that will be carried out will be of NDT type, that is non-destructive tests: as a cultural heritage asset, the church may be subjected to tests, like thermography, and sonic tests, that will produce no damage.

These diagnostic tests will then be useful to proceed with numerical analyses. The analyses performed are intended to help understand the behaviour of the structure under seismic conditions, with a standard return period. The area in which the analysed structure is located is characterized by low seismic levels, in fact even the design response spectrum is not very demanding, but churches are structures easily damaged in earthquakes, so it is nevertheless useful to proceed with initial linear analyses to understand what safety level the structure has, and if it requires interventions.

The analyses performed have progressive order of complexity and concern single parts of the building up to the entire structural model. These analyses have allowed a first assessment of the seismic response, indicating the parts most exposed to damage and a global safety level of the asset.

SOMMARIO

L'obiettivo di questa tesi di laurea magistrale è la valutazione del danno e delle condizioni strutturali di un edificio storico in muratura, valutandone il comportamento sismico atteso per il livello di progetto considerato per il sito in Provincia di Lodi. Il caso di studio è la Chiesa dell'Assunzione della Beata Vergine Maria a Villavesco, Tavazzano.

Le analisi vengono effettuate studiando inizialmente la struttura dal punto di vista geometrico, procedendo con analisi che permetteranno di valutare la composizione della struttura, gli eventuali danni, e le condizioni attuali di danno della struttura stessa. Ovviamente, le analisi che verranno effettuate saranno di tipo NDT, quindi analisi non distruttive: in quanto bene culturale, la chiesa potrà essere sottoposta a test, come la termografia o le prove soniche, che non produrranno danni. Questi test diagnostici saranno poi utili per procedere con analisi numeriche.

Le analisi eseguite hanno lo scopo di aiutare a comprendere il comportamento della struttura in condizioni sismiche, con un periodo di ritorno standard. L'area in cui si trova la struttura analizzata è caratterizzata da bassi livelli di sismicità; infatti, anche lo spettro di risposta progettuale non è molto impegnativo, ma le chiese sono strutture facilmente danneggiabili in caso sismico, quindi è comunque utile procedere con analisi lineari iniziali per capire quale livello di sicurezza ha la struttura, e se sono necessari interventi. Le analisi eseguite sono in ordine progressivo come complessità e si estendono da parti dell'edificio fino all'intero modello strutturale.

Queste analisi hanno permesso una prima valutazione della risposta sismica, indicando le parti più esposte ai danni e un livello di sicurezza globale del bene.

INDEX

INDEX OF FIGURE 12

INDEX OF TABLE 15

1. THE CASE STUDY 20

1.1. Location 20

1.1.1. Definition of the soil characteristic 23

1.2. The church 25

1.2.1. The story 25

1.2.2. Restauration 25

1.2.3. Geometric Description 26

1.2.4. Description of the main structural components 29

1.2.5. Current Building Conditions 33

2. SEISMIC DAMAGE AND VULNERABILITY 37

2.1. Seismic Risk 37

2.2 Mechanical model for the analysis of churches 38

2.3 Damage Limit Mechanisms 40

2.4 Model A-DC 46

2.4.1 Damage mechanism of the case study 47

2.5 Characteristics of the model for analysis 49

2.6 Confidence Factor for a masonry building 52

2.7 Non-destructive tests for the masonry 53

3 IN SITU TESTS 54

3.1. Thermography 55

3.1.1. Results 57

3.2. Sonic and ultrasonic test 59

3.2.1. Wall thickness measurement 61

3.2.2. Results of the sonic test 63

4 **LOADS ANALYSIS** 66

2.2 Evaluation of the seismic action "E" 67

4.1.1 Nominal Life, Class of Use and Reference Period 70

4.2 Design Spectrum 72

4.2.1 Behaviour factor "q" 73

4.2.2 Design Spectra, NTC-Excel 75

5 **GEOMETRIC MODELING** 80

5.1 Masonry 82

5.2 The roof 83

5.3 The bell tower 84

5.4 The vaults 85

5.5 Render of the complete architectonic model 86

6 **FEM MODELING** 89

7 **ANALYSIS** 93

7.1 Static Analysis 93

7.1.1 Structural Wall and Façade 93

7.1.2 Bell Tower 94

7.1 Modal Analysis 95

7.1.1 Church (Structural Wall, Arches and Façade) 95

7.1.2 Bell Tower 98

7.1.3 Church (Structural Wall, Arches, Façade and Tower) 107

7.1.4 Complete Model of the Church 110

7.2 Response Spectrum Analysis 116

7.2.1 Church (Structural Wall, Arches and Façade) 116

7.3 Results 117

7.3.1 Bell Tower 118

8. LIMIT ANALYSIS 120

8.1 Linear cinematic analysis of the tympanum behaviour 120

9. CONCLUSIONS 125

INDEX OF FIGURES

Figure 1 - Location of the Case Study 20
Figure 2 - Teresian Cadaster, 1722, Villavesco 21
Figure 3 - Lombardo-Veneto Cadaster, 1860, Villavesco 21
Figure 4 - Earthquake 1987, Reggiano 22
Figure 5 - Structural Plan 26
Figure 6 – The church 28
Figure 7 - Internal central nave view 29
Figure 8 - Interior of the bell tower 30
Figure 9 – Roof Structure 31
Figure 10 - View of the Church from the Frontal Facade 32
Figure 11 - The bell tower 33
Figure 12 – Conservation through humidity reduction systems 34
Figure 13 – External Walls Defects 35
Figure 14 - Protection against plaster detachment 36
Figure 15 - Schematization of macro-elements of historic building [NOTA] 40
Figure 16 - Damage Mechanism 45
Figure 17 - Cracking mechanism of the façade 47
Figure 18 - Side wall cracking mechanism 47
Figure 19 - Meccanismo di fessurazione della cella campanaria 48
Figure 20 - Meccanismi di fessurazione della torre campanaria 48
Figure 21 - Levels of knowledge and confidence 50
Figure 22 - Parameters for different typology of masonry 51
Figure 23 - Location of the tests carried out 54
Figure 24 – Instrument used and first thermography results 56
Figure 25 – Test T07, South East side and Bell Tower 57
Figure 26 – Difference of materials inside the bell tower and thermography 57
Figure 27 - Test T09, Thermographic images, Southwest elevation 58
Figure 28 – Scheme of sonic test modalities 59
Figure 29 - Illustration of the methodology 60

Figure 30 - Instrument for the wall thickness measurement 61
Figure 31 - Photographic representation during in situ testing 62
Figure 32 – Photographic representation during in situ testing 62
Figure 33 – Test S01, Facade 63
Figure 34 – Test S04, Bell Tower 64
Figure 35 – S05 Test, Apse 65
Figure 36 - Seismic hazard model MPS04-S1, INVG 67
Figure 37 - Ground acceleration map for Tavazzano con Villavesco 68
Figure 38 - Shaking map for case study in Tavazzano con Villavesco 68
Figure 39 - Eurocode 8, Design Respose Spectrum for different types of soil 72
Figure 40 - Tab 7.2.II Maximum values of the base value q0 of the SLV 73
Figure 41 - Tab. 3.2.II 75
Figure 42 - Tab. 3.2.III – Categorie topografiche 75
Figure 43 - Bell Tower, Top View 84
Figure 44 - Bell Tower Complete Model 84
Figure 45 - Architectonic Model, Structural Wall and Facade 86
Figure 46 - Architectonic Model, Structura Wall, Facade and Bell Tower 86
Figure 47 - Architectonic Model with the vaults 87
Figure 48 - Architectonic Model with the Structure of the Wood Roof 87
Figure 49 - Complete Model, Frontal View 88
Figure 50 - Final Architectonic Model, Rhino 3D 88
Figure 51 - Block 1 imported in Abaqus 89
Figure 52 - Block 2 imported in Abaqus 90
Figure 53 - Boundary Condition - Assembly Block 1 and Block 2 90
Figure 54 - Gravity Loads applied to the model 91
Figure 55 – Mesh Creation 92
Figure 56 – Static Analysis, S Mises 93
Figure 57 – Static Analysis, U3 93
Figure 58 – Static Analysis, Bell Tower, S33 94
Figure 59 – Static Analysis, Bell Tower, Mises 94
Figure 60 – Modal Analysis, Mode 1 95

Figure 61 – Modal Analysis, Mode 2......96
Figure 62 – Modal Analysis, Mode 3......97
Figure 63 – Modal Analysis, Mode 4......97
Figure 64 – Modal Analysis, Bell Tower, Mode 1......99
Figure 65 – Modal Analysis, Bell Tower, Mode 2......99
Figure 66 – Modal Analysis, Bell Tower, Mode 3......100
Figure 67 – Modal Analysis, Bell Tower, Mode 4......100
Figure 68 – Modal Analysis, Bell Tower, Mode 5......101
Figure 69 – Modal Analysis, Bell Tower, Mode 6......101
Figure 70 – Modal Analysis, Bell Tower, Mode 7......102
Figure 71 – Modal Analysis, Bell Tower, Mode 8......102
Figure 72 – Modal Analysis, Bell Tower, Mode 9......103
Figure 73 – Modal Analysis, Bell Tower, Mode 10......103
Figure 74 – Modal Analysis, Church - Bell Tower, Mode 1......108
Figure 75 – Modal Analysis, Church - Bell Tower, Mode 2......109
Figure 76 – Modal Analysis, Church - Bell Tower, Mode 3......109
Figure 77 – Modal Analysis, Complete Model, Mode 1......111
Figure 78 – Modal Analysis, Complete Model, Mode 2......111
Figure 79 – Modal Analysis, Complete Model, Mode 3......112
Figure 80 – Modal Analysis, Complete Model, Mode 4......112
Figure 81 – Modal Analysis, Complete Model, Mode 5......113
Figure 82 – Modal Analysis, Complete Model, Mode 6......113
Figure 83 – Modal Analysis, Complete Model, Mode 7......114
Figure 84 – Modal Analysis, Complete Model, Mode 8......114
Figure 85 – Modal Analysis, Complete Model, Mode 9......115
Figure 86 – Modal Analysis, Complete Model, Mode 10......115
Figure 87 – Response Spectrum, Church......116
Figure 88 – Response Spectrum, Church......116
Figure 89 – S. Max and S. Min Principal, Bell Tower......118
Figure 90 – S. Max and S. Min Principal, Bell Tower......119

INDEX OF TABLES

Table 1 - Subsurface categories that allow the use of the simplified approach 23

Table 2 - Categorie topografiche, Tab. 3.2.III - NTC 24

Table 3 – Guide Lines for Cultural Heritage Tab 4.1 - Chapter 4.2 52

Table 4 – Nominal Life according to the construction, Tab. 2.4.I NTC 70

Table 5 – V_R values depending on V_N and C_U, Tab. C2.4.I - NTC 71

Table 6 - Probability PVR depending on the limit state, Tab. 3.2.I - NTC 71

Table 7 - Modal Analysis, Structure & Tower 107

INTRODUCTION

Seismic events are a primary factor of risk for the cultural heritage in the Italian territory. Ancient masonry churches are particularly subjected to seismic damage, due to their construction characteristics. This thesis studies a historical masonry church, assessing first its structural characteristics and current conditions in terms of possible decay and initial damage and finally estimating its expected seismic behaviour for the level of design earthquake considered for the site. The case study is the church of Assunzione della Beata Vergine Maria in Villavesco, Tavazzano.

In the first chapter a brief description of the case study, the site and the main features will be made; this step is fundamental to understand the relevance of the building concerned, to understand its characteristics, weaknesses, and the main problems to which it could be subjected. The study and determination of the characteristics of the case study are also important because they will be needed in subsequent chapters for seismic analysis.

After a general study of the church, some tests will be carried out to allow to assess the composition of the structure, possible damage, and current conditions of the structure itself. Obviously, the analyses that will be carried out will be NDT type, thus non-destructive analyses; as a cultural heritage asset, the church may be subjected to tests, like thermography, and sonic tests, that will produce no damage. These diagnostic tests will then be useful to proceed with numerical analyses.

The analyses performed are intended to help understand the behaviour of the structure under seismic conditions, with a standard return period.

The area in which the analysed structure is located is characterized by low seismic levels, in fact even the design response spectrum is not very demanding, but churches are structures easily damaged in earthquakes, so it is nevertheless useful to proceed with initial linear analyses to understand what safety level the structure has, and it is requires further investigations and interventions.

The analyses performed have progressive order of complexity and concern single components of the building up to the entire structural model. These analyses have allowed a first assessment of the seismic response, indicating the parts most exposed to damage and a global safety level of the asset.

1. THE CASE STUDY

The Church of Tavazzano-Villavesco, a parish church dedicated to " Assunzione della Beata Vergine Maria", is the object of study of this thesis; the church is located in the village of Tavazzano con Villavesco, in the province of Lodi; the church is in the town center and is integrated with other historic buildings. With the aim of creating a three-dimensional model suitable for a reliable seismic analysis, attention is paid not only to today's characteristics but, where possible, also to the changes made in the past and to the events that such historic building has undergone.

1.1. Location

The case study in question is in the Municipality of Tavazzano con Villavesco, in the province of Lodi, in Lombardy. The city is located at 80 meters above sea level in the Po river Plain. The territory is crossed by the river Sillaro and has a regular altimetric profile, with minimal variations. Tavazzano and Villavesco were considered for long two distinct villages. They were consolidated in 1869, and also the two names were joined; initially the small town was built on the banks of the Sillaro, near the hospital "Sanctae Mariae et Johannis Baptista de Tavazzano" (built by Modalino in 1170). After the disappearance of the hospital, due to the annexation of the same to the Maggiore Hospital of Lodi, the village lost its importance.

Figure 1 - Location of the Case Study

Over time some transformations of the volumes of the church have taken place, and the historical land registers contain the main information. Below are some of the fundamental extracts taken from the most relevant land registers, which mark the evolution of our case study. The following image shows an extract from the Teresian Cadaster, made around 1722; at that time the parish was already present, except for the semicircular apse.

Figure 2 - Teresian Cadaster, 1722, Villavesco

With the subsequent Lombardo-Veneto Cadaster, dating back to about 1860, it can be noted that the volumes of the building considered have changed considerably; in fact, the apse and part of the new nave to the east are added.

Figure 3 - Lombardo-Veneto Cadaster, 1860, Villavesco

As for its seismic history, one of the strongest earthquakes that struck this town was the earthquake in May 1987, with its epicenter in Reggio Emilia; the intensity level of the epicenter was between 6-7 and came to the church with intensity equal to 5.

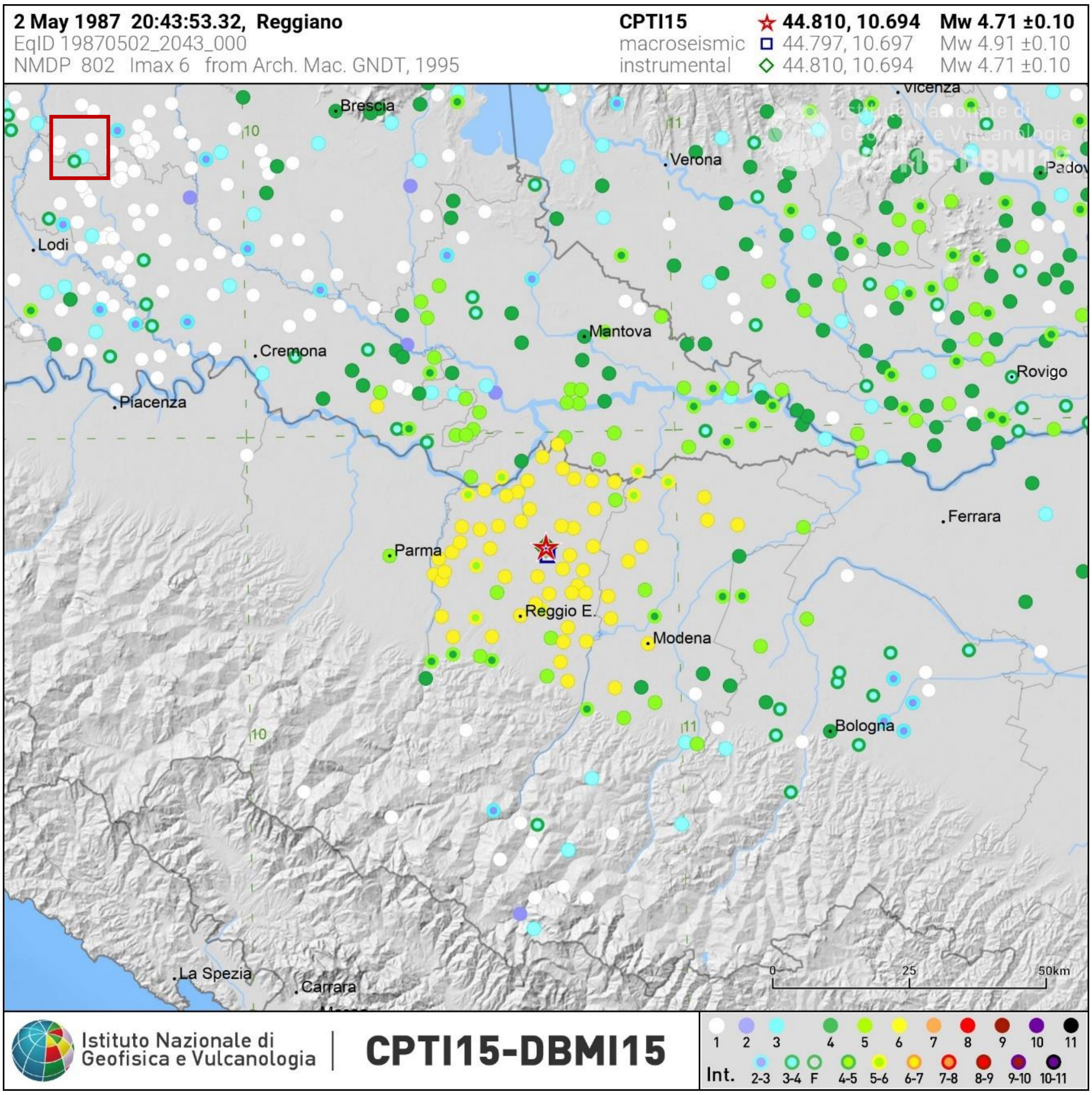


Figure 4 - Earthquake 1987, Reggiano

Another earthquake occurred in 2002, with its epicenter in Franciacorta, but this did not cause damage and also the level of perceived intensity was much lower.

Similarly, no significant effects were observed from the Pianura Padana emiliana Earthquake of 2012.

1.1.1. Definition of the soil characteristic

The soil is a fundamental part of the analysis because it has a very important impact on the structural behavior of the building under analysis.

As we will see later, in fact, the seismic response spectrum depends on the category of the soil, for this reason in the Technical Standards of Construction (NTC 2018) different types of soil are defined, shown in the following table.

This is a simplified approach that is based on the classification of the subsurface according to the speed of propagation of the shear waves.

Table 1 - Subsurface categories that allow the use of the simplified approach

Categorie	**Descrizione**
A	*Ammassi rocciosi affioranti o terreni molto rigidi* caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	*Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti*, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 e 800 m/s.
C	*Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti* con profondità del substrato superiore a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi 380 e 360 m/s.
D	*Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fine scarsamente consistenti,* con profondità del substrato superiori a 30m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D, con la profondità del substrato non superiore a 30m.

The category of soil should be defined after performing tests to measure the s-waves velocity or, at least, carrying out static geotechnical tests. In this study, such tests could not be performed. In the case study considered the category of soil may be reasonably assumed as "C", which is a medium soil according to the characteristics described.

In addition, according to the NTC it is also possible to define the topographic category, as show in the table below.

At the site, the soil surfaces is flat, corresponding to T1.

Table 2 - Categorie topografiche, Tab. 3.2.III - NTC

Categorie	**Caratteristiche della superficie topografica**
T1	Superficie pianeggiante, pendii e rilievi isolati con **inclinazione** media 15°
T2	Pendii con inclinazione media i > 15°
T3	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media 15° < i < 30°
T4	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media i > 30°

1.2. The church

1.2.1. The story

The architecture of the "Chiesa dell'Assunzione della Beata Vergine Maria", as can be seen from the plan, recalls the period of the Counter-Reformation. Thanks to the documentation found at the site of the Authority for cultural Heritage of Lombardy, it is possible to trace some fundamental dates relating to the changes, which took place over time, of the church under analysis. Unfortunately, the first information does not concern the construction of the church itself, but the construction of the baptistery, requested in 1674 by the rector of Villavesco to the Episcopal Curia of Lodi. In fact, he asked to remove the baptistery from its initial position (the current Chapel of Our Lady of Sorrows) and place it in a new chapel, not yet built at that time.

Another piece of information dates to the year 1695 where the chapel of the Blessed Virgin of the Rosary on the right side was mentioned, for the first time.

The construction of a new choir took place in 1733, commissioned to Giovanni Giudici, to replace the wooden one, that was ruined. The altar is indicated as new in a document dated January 22, 1734. In other documents, it is described since 1770 as composed of a circular tabernacle, supported by eight decorated marble columns. The altar area and the choir area are separated by the presence of two pilasters. As for the decoration of the walls and the presbytery, the Lodi painter Silvio Migliorini took care of it, the date of the frescoes is not currently known.

1.2.2. Restauration

Even if older interventions on the church structure are not known, it is always very important to also look at the changes that have been made in the recent past, to evaluate the effects of any modifications:

- Conservative restoration of the bell tower, 1980
- Consolidation and refinement of walls, 1985
- Consolidation of the façade and renovation of the plaster, 1993-1998
- Ordinary maintenance of roofing surfaces, replacement of asbestos-cement slabs with fiber cement slabs, 2020 – 2022
- Renovation of damaged plaster of the bell tower, 2020 – 2022.
- (further interventions on the roof are in progress)

1.2.3. Geometric Description

Structural Plant

The structure is typical of Christian basilicas with a longitudinal plan, rectangular in shape. As can be seen from the plan below [*Figure 5*], which represents the actual situation of the church, the entrance is placed directly on the road, elevated above it thanks to a few steps and a porch.

The figures shown below refer to surveys made available by the Diocese of Lodi; as will be shown later, some checks were carried out punctually to evaluate the existing structure in terms of thickness and materials. The Church of the Assumption of Villavesco does not have a transept, as it is of Christian basilicas.

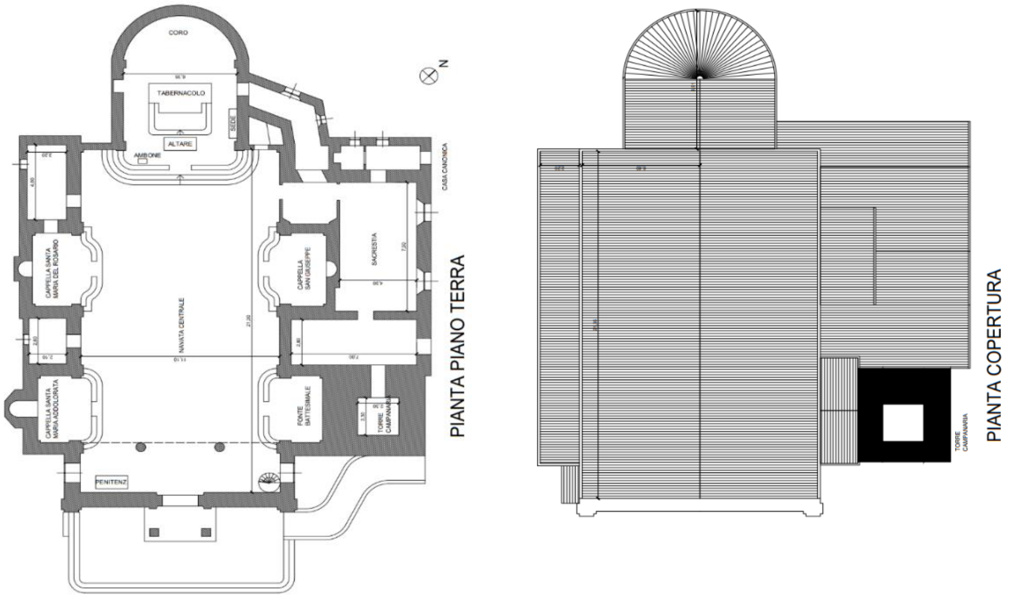


Figure 5 - Structural Plan

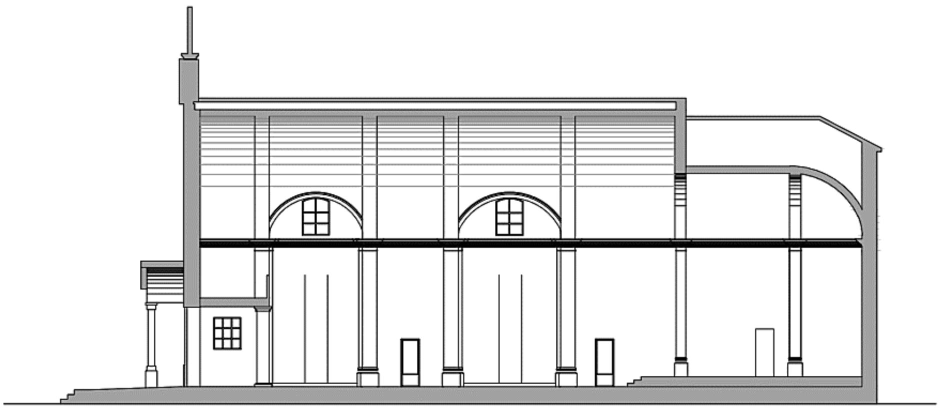


Figure 6 – Longitudinal Section

Elevation Plan

Figure 7 - Elevation South East

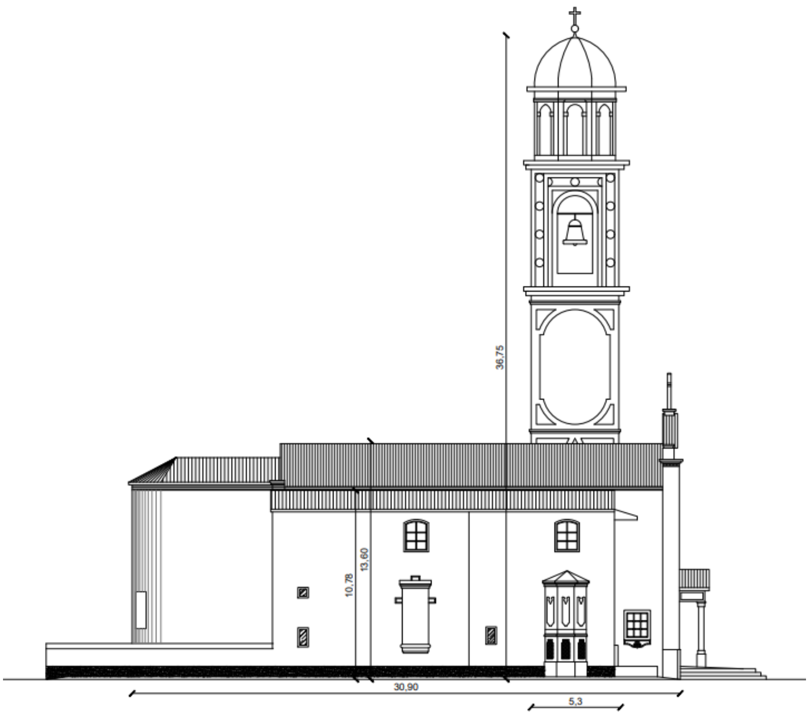


Figure 8 - Elevation South-West

Figure 6 – The church

1.2.4. Description of the main structural components

Central Nave & Sacresty

The central nave has modest dimensions of 21.00m and 11.10m; there are four side chapels that have a similar size and are symmetrical with each other two by two; on the North-East side we find the sacristy, again with a rectangular shape; a series of arches divides the length of the vault that covers the nave, the side chapels have a barrel vault, that intersects the vault of the nave. For the construction techniques, there is the presence of vertical walls in brick masonry plastered with decorations, as regards the church and the sacristy.

The sacristy is located on the right side of the church and is accessible both internally and externally. It also has a rectangular plan and the roof shows two cross vaults, generated by the intersection of barrel vaults.

At the end of the nave, as can be seen from the image, there is a rectangular apse and a semicircular choir; these are covered by a spherical cap, internally decorated with three-dimensional representations.

Figure 7 - Internal central nave view

Bell Tower

The bell tower is located on the right side of the church and it can be reached internally from the sacristy; this is located to the right of the church and is also accessible externally from the courtyard; it has a rectangular development covered by several barrel vaults that, intersected with each other, generate a double cross vault.

The bell tower is composed by a continuous and visible brick masonry, with wood slabs and stairs that allow to pass from one floor to another, in case of necessity. Inside the bell tower you can also see the presence of some cross vaults, present when the bell tower was still in its original position. Internally the bell tower is plastered up to about 2cm in height, then the masonry is exposed.

Figure 8 - Interior of the bell tower

As for the opening, the roof is a simple roof with two symmetrical pitches, with a truss as the primary structure, the cladding is made of tiles.

The bell tower was not originally in this position but was later rebuilt in continuity with the church. For this reason, in the current area where the tower stands there are stretches of some cross vaults, therefore present before the move.

The old layout of the rooms is visible because many openings are not symmetrical, and some doors were walled up.

Recently, restoration work has been carried out on the church roof; this is visible from the attic and the openings of the bell tower. A ridge beam is supported by masonry pillars that are positioned at the end of the vaults that we find in the nave; from here, orthogonally, other joists are distributed and supported by the perimeter walls.

Figure 9 – Roof Structure

Façade

The façade consists of a rectangular proteron, with a triangular tympanum located above the entrance door; the proteron is elevated above the base floor of the street. The eardrum is supported by two stone columns.

The church is well integrated with other buildings; the external façade is plastered unevenly and at the upper order there is a bas-relief above the terracotta porch as a Eucharistic symbol, two side windows with a decorative and illustrative panel in the center. At the lower order of the façade, we find two niches with statues of Saints, symmetrical with respect to the central door.

Even the bell tower is decorated externally even if, lately, it is subject to degradation and detachment of some parts of plaster; here we find a frame with geometric patterns.

Figure 10 - View of the Church from the Frontal Facade

1.2.5. Current Building Conditions

External Damage

One of the most damaged points of the exterior is the bell tower, where cracks and detachment of plaster are clearly visible. The main façade, facing South-East, is the side that presents the greatest problems, the degradation has developed points where the wall is totally exposed. The south-facing side is the one with the least damage.

Figure 11 - The bell tower

The bell tower, as previously confirmed, presents decay on each side, in fact it has detachment of plaster in various points and serious cracks, in the most exposed facades there is the lack of even a few square meters of plaster; there is therefore the risk of the sudden detachment of some portions of the cover of the wall.

Other defects visible externally are to be attributed to the presence of humidity, given the exposed area, there are therefore many significant phenomena of capillary ascent.

The problem of humidity is very important and must be kept under constant control; the causes of moisture formation can concern external and internal factors; external factors mainly concern water infiltration and rising from the subsoil, while internal ones depend on air humidity and therefore the presence of condensation on surfaces.

To avoid aesthetic damage and try to reduce the danger of these damages, slabs have been placed at the foot of the building that cover up to a height of about 70cm from the ground. On the façade and on the back, however, "vents" have been placed that are clearly noticeable as they have a diameter of about 10 to 15cm. They are visible as pipes, covered externally by grids, which have the function of reducing the accumulation of moisture inside the masonry.

Figure 12 – Conservation through humidity reduction systems

These interventions have been made very recently on this structure, it is thought in different processes as the pipes differ in the type of material used, plastic or aluminum with covering grids. Generally, this system is widely used in this area, since it is in the Pianura Padana Region, and so the presence of moisture is the normality.

Other types of external damage concern water leaks on the roof groundwater, leakage from pipes and the surface formation of condensation; The damage is not structural and therefore does not damage the structure itself, but aesthetic damage in the form of very dark stains, detachment of plaster is evident.

On the side of the inner courtyard, however, where there is the parish priest's house, the damage is much more evident and concerns the structural part of the building.

Figure 13 – External Walls Defects

This very evident damage is reminiscent of the presence and accumulation of pollutants on the surfaces; due to the lack of protective elements, as the walls are totally exposed to atmospheric agents and rainwater discharges.

Internal Damage

Compared to the outside, the internal damage is milder, although the presence of mold and small sediments of external material can be found, due to polluting external elements. Currently (as clearly visible in the figure) in the central nave protection systems have been mounted as lately there have been episodes of detachment of plaster from the ceiling. As can be seen again in the figure, there are also horizontal support elements, tie-rods, useful to join the two sides of the nave, which retain the thrust from arches but are also an important protective element in case of earthquake.

Figure 14 - Protection against plaster detachment

Inside the church it is also possible to notice the presence of many cracks that are found on most of the walls; for this reason, it was chosen to proceed with some investigations to characterize the walls, with the aim of knowing the materials and any unevenness.

2. SEISMIC DAMAGE AND VULNERABILITY

2.1. Seismic Risk

The damage that usually occurs to historic masonry buildings indicated the need to assess their seismic vulnerability and, if required, perform interventions for their seismic improvement, in order to reduce seismic risk.

To show the theory concerning the assessment of seismic vulnerability, we consider in this chapter the Italian Legislation, NTC 2018 and its previous one of 2008. This provides for and establishes the criteria necessary for the assessment of safety on existing buildings, that is, constructions that have the structure completely realized. The methods of analysis and verification that are presented in this legislation will be considered, considering that the safety assessment returns a quantitative parameter.

The different limit mechanisms and the A-DC model, for damage assessment in churches after an earthquake, will then be presented, which represents the current basis used to detect damage in historic churches. This has been widely used in recent years, after the earthquakes that occurred in central Italy since the end of the 20th century.

As a general frame of the problem, it is important to recall the definition of risk. The *seismic risk* may be intended as a measure of the damage level deriving from possible earthquakes, usually expressed in probabilistic terms.

The seismic risk basically depends on three main parameters, which are respectively:

- P, Seismic Hazard, which is the probability of a specified level of ground motion at a specific location.
- V, Vulnerability, which is a measure of a constructions to withstand seismic actions.
- E, Exposure, which may be defined in different ways depending on the contest, but basically represents value.

Of course, the risk, as by its definition, cannot be eliminated, but it can be reduced by focusing on some elements that contribute to it, basically on vulnerability.

2.2 Mechanical model for the analysis of churches

For the analysis of the global mechanisms of damage for the structure, in the case of structures, different methods can be used as shown below:

- *Dynamic modal analysis,* this type of analysis, in the linear field, is widely used. It identifies the main modes of vibration in each direction of the system with reference to a response spectrum the stress levels of the structural elements, and the structure may be verified. Therefore, it allows to calculate all the modes of vibrating of the structure under analysis, the period of vibration associated with each mode and the participating mass for each mode of vibration.
- *Nonlinear static analysis*, this type of analysis considers the non-linearity of the materials and, unlike the linear one, evaluates the behavior of the structure, analyzing the force-displacement law, and finding the displacement capacity in SLU condition, close to the activation of the collapse mechanism. Checks can be performed by considering a model that represents the overall behavior of the structure. This analysis consists in the application of gravitational loads and a system of horizontal forces that are scaled until the ultimate conditions are reached.
- *Nonlinear dynamic analysis*, this type of analysis consists in the calculation of the seismic response by integrating the equations of motion using a nonlinear model of the structure. To use this type of analysis it is necessary to consider different types of accelerograms, which must be compatible with each other with the response spectrum with respect to the type of subsoil. This analysis is therefore used and recommended in very particular conditions; it is not a standard analysis that is always carried out.

Referring to the local limit conditions, that is, when a limited portion of the building, may be damaged and evolve into collapse, it is necessary to recognize a possible kinematic chain

- *Linear kinematic analysis* computes the horizontal multiplier of the loads that activate the collapse mechanism based on the initial configuration of the chain; from this, the evaluation of the corresponding seismic action may be performed.

- *Non-linear kinematic analysis* includes the variation of the geometry while the mechanism develops; it is more precise than the frmer one and is used for tìfurther verification once the linear analysis gives negative results.

In the linear kinematic analysis, through the P.L.V it is possible to evaluate the horizontal multiplier by the following formula, considering the different weight forces:

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

Where:

- "*m*" is the number of weight forces not directly imposed on the blocks whose masses generate horizontal forces on the elements of the kinematic chain, as they are not effectively transmitted to other parts of the building.
- "*n*" is the number of weight forces applied to the different blocks.
- "*o*" is the number of external forces applied to the different blocks.
- "P_i" is the generic weight force applied.
- "P_j" is the generic weight force, not directly applied on the blocks, whose mass generates a horizontal force on the elements of the kinematic chain.
- "$\delta_{x,i}$"is the horizontal virtual shift of the point of application of the i_{-th} weight Pi.
- "$\delta_{x,j}$"is the horizontal virtual shift of the point of application of the j_{-th} weight Pj.
- "$\delta_{y,i}$"is the vertical virtual shift of the point of application of the i_{-th} weight Pi.
- "F_h is the generic external force.
- "L_{fi}" is the work of internal forces, where present.
- "δ_h" is the virtual displacement of the point where the h_{-th} external force is applied, in the same direction.

The forces that are considered are therefore those weighing on the kinematic structure, the weight forces not directly burdened, the external forces, the internal forces and then considering the different displacements after imposing a rotation. In the linear analysis of loads, considering rigid bocks, the internal force amount to 0. After the activation of the mechanism, one can consider the spectral acceleration that is provided by the horizontal seismic force of activation, divided by the considered mass:

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^*}$$

The mass "M" is evaluated considering all the virtual displacements at the respective points of application.

2.3 Damage Limit Mechanisms

The damage mechanisms that are observed in a church after an earthquake are recurrent and are recognizable as each component of the church has its own specific mode of damage; to facilitate the observation of these modalities, especially for the brick masonry church, the concept of "macro-element" was introduced, that is, an architectural element or part that is characterized by an autonomous seismic response with respect to the rest of the components. Thanks to in-depth analysis over the years, through the study of damage and severe events that happened in the past, it is possible to recognize how the macro-elements, identifiable in the structure, tend to show the behavior of a rigid body; thus, a macro-element is also identifiable in that it behaves independently of the rest of the structure.

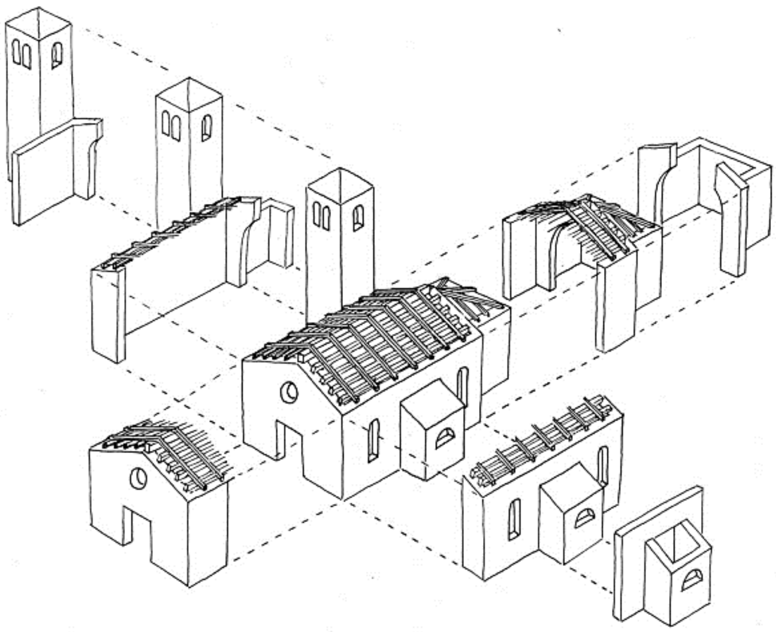


Figure 15 - Schematization of macro-elements of historic building [NOTA][1]

[1] "Istruzioni Tecniche per l'interpretazione ed il rilievo per macroelementi del danno e della vulnerabilità sismica delle chiese – Evento sismico del 01.04.2000 - Ordinanze Ministero

The behavior of these macro-elements was studied and published by the Prime Ministerial Decree of 23 February 2006 and subsequently on 9 February 2011, where a "Model A-DC" card is proposed.

Following the logic set by the Guidelines of Cultural Heritage (OJ 2011), we proceed with a subdivision of the church into macro-elements; the structure of the church is divided into a finite number of macro-elements, which are 9, and in total 28 damage mechanisms are proposed, divided between the different types of macro-elements; the different typology of macro-element that we can have, generally are:

- The façade
- The triumphal arch
- The transept
- The dome
- The bell towers
- The porch
- The nave
- The apse
- The chapels

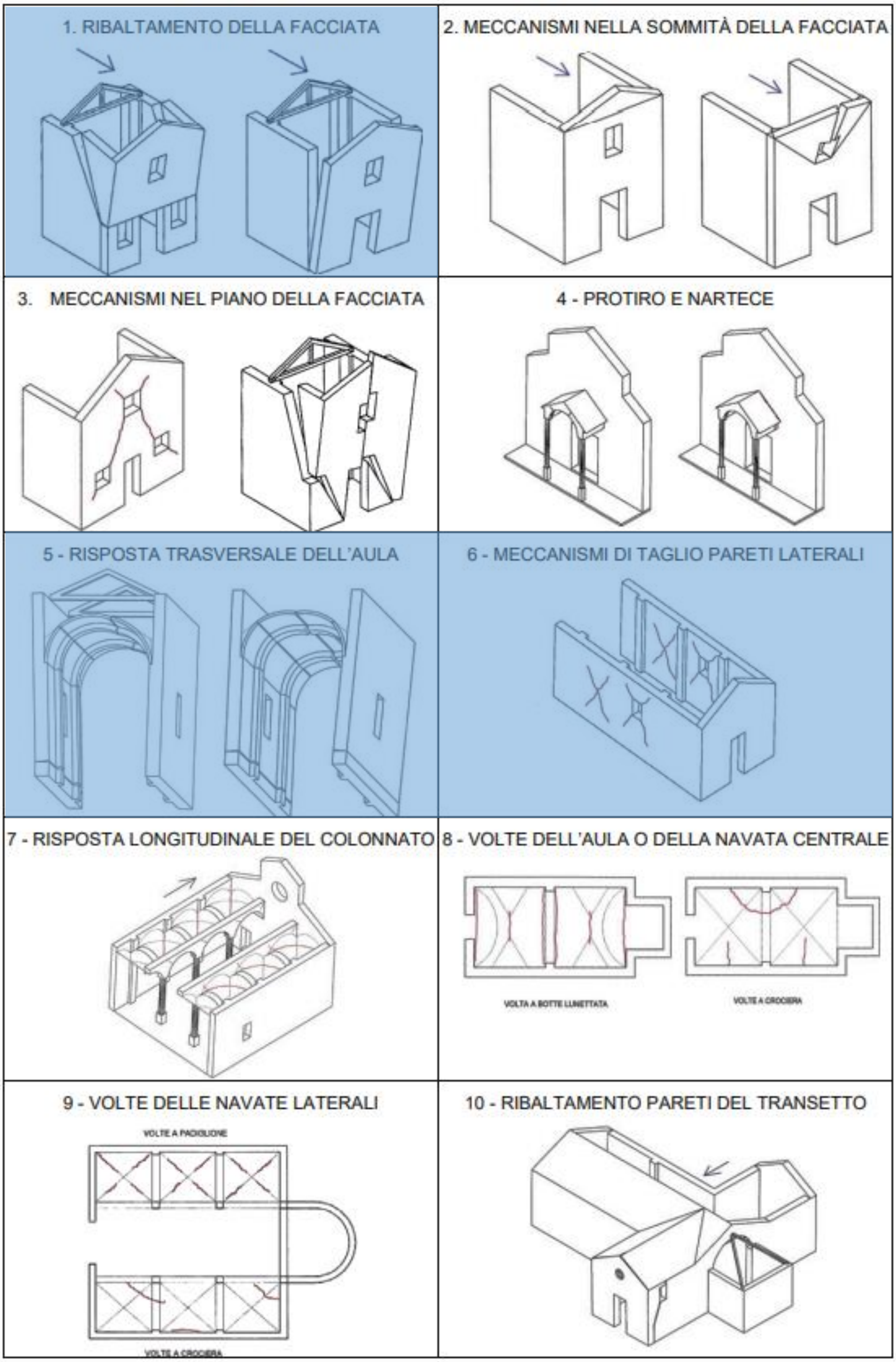
Using a correct model, it is possible to evaluate the vulnerability index, starting first of all from the subdivision of the macro-elements.

As stated in the "*Allegato C - Modello per la valutazione della vulnerabilità sismica delle chiese*[2] before proceeding, it is necessary to check the presence of macro elements, because obviously not everyone could always be present, and consequently understand which damage mechanisms are already negligible.

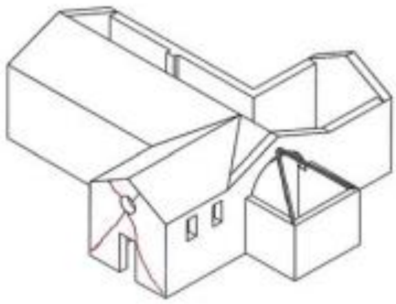
[2] *Supplemento ordinario n. 54 alla GAZZETTA UFFICIALE* dell'Interno Dip. Protezione Civile n°3061 del 30.06.2000, n° 3124 del 12.04.2001 e n° 3146 del 15.08.2001, Allegato A".

Below is the table with the macro elements and their respective mechanisms:

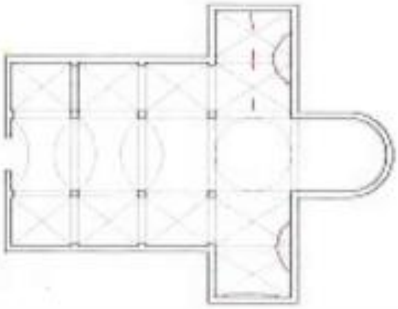
Mechanism of damage	**Microelement**
M1 – Ribaltamento della facciata	Facciata
M2 – Meccanismi nella sommità della facciata	
M3 – Meccanismi nel piano della facciata	
M4 – Protiro e narcete	
M5. Risposta trasversale dell'aula	Aula
M6. Meccanismi di taglio delle pareti laterali	
M7. Risposta longitudinale del colonnato	
M8. Volte dell'aula e della navata centrale	
M9. Volte delle navate laterali	
M10. Ribaltamento delle pareti del transetto	Transetto
M11. Meccanismi di taglio del transetto	
M12. Volte del transetto	
M13. Archi trionfali	Arco trionfale
M14. Cupola e tamburo/tiburio	Cupola
M15. Lanterna	
M16. Ribaltamento dell'abside	Abside
M17. Meccanismi di taglio nell'abside	
M18. Volte del presbiterio o dell'abside	
M19. Meccanismi negli elementi di copertura - pareti laterali dell'aula	Copertura
M20. Meccanismi negli elementi di copertura - transetto	
M21. Meccanismi negli elementi di copertura - abside	
M22. Ribaltamento delle cappelle	Cappelle e corpi annessi
M23. Meccanismi di taglio nelle pareti delle cappelle	
M24. Volte delle cappelle	
M25. Interazioni in prossimità di irregolarità plano-altimetriche (corpi adiacenti, archi rampanti)	
M26. Aggetti (Vela, guglie, pinnacoli, statue)	
M27. Torre campanaria	Aggetti campanili
M28. Cella campanaria	


1. RIBALTAMENTO DELLA FACCIATA
2. MECCANISMI NELLA SOMMITÀ DELLA FACCIATA
3. MECCANISMI NEL PIANO DELLA FACCIATA
4 - PROTIRO E NARTECE
5 - RISPOSTA TRASVERSALE DELL'AULA
6 - MECCANISMI DI TAGLIO PARETI LATERALI
7 - RISPOSTA LONGITUDINALE DEL COLONNATO
8 - VOLTE DELL'AULA O DELLA NAVATA CENTRALE
VOLTA A BOTTE LUNETTATA
VOLTE A CROCIERA
9 - VOLTE DELLE NAVATE LATERALI
VOLTE A PADIGLIONE
VOLTE A CROCIERA
10 - RIBALTAMENTO PARETI DEL TRANSETTO

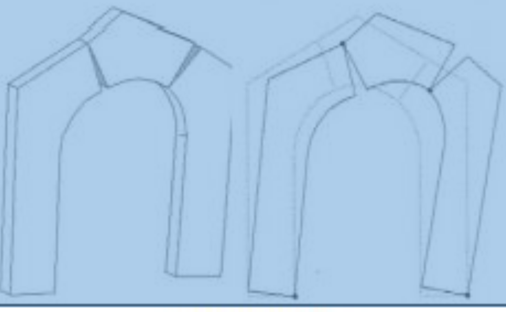
11 - MECCANISMI DI TAGLIO DEL TRANSETTO



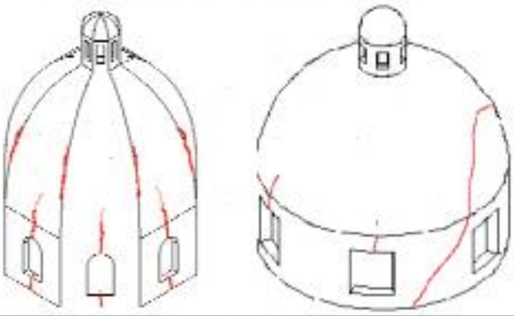
12 - VOLTE DEL TRANSETTO



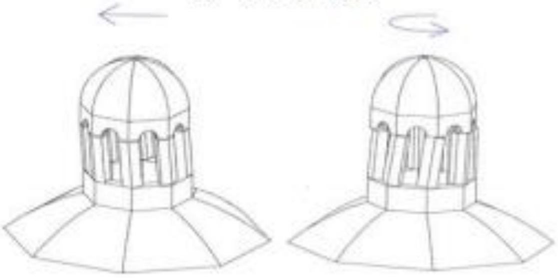
13 - ARCHI TRIONFALI



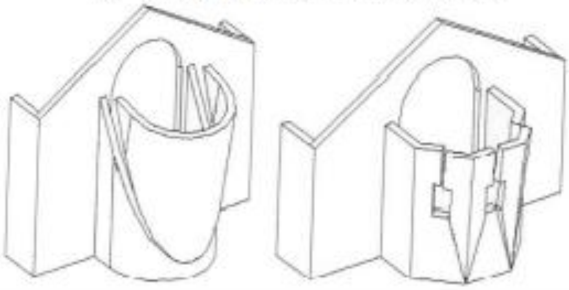
14 - CUPOLA E TAMBURO / TIBURIO



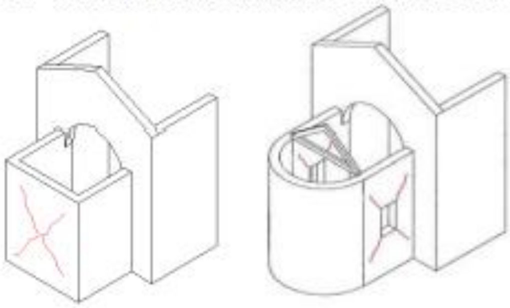
15 – LANTERNA



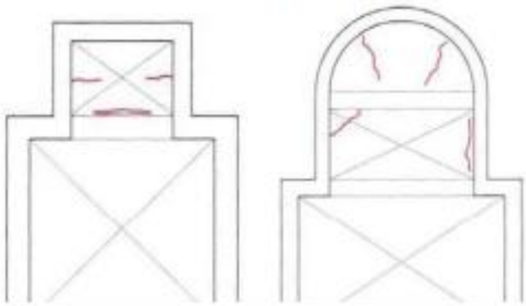
16 - RIBALTAMENTO DELL'ABSIDE



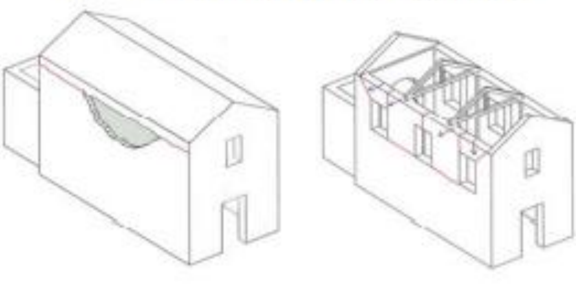
17 - MECCANISMI DI TAGLIO NELL'ABSIDE



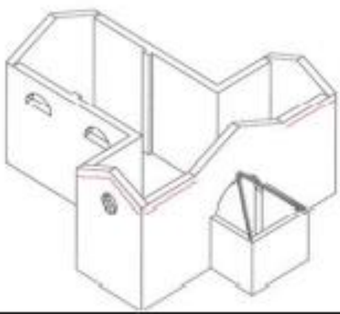
18 - VOLTE DEL PRESBITERIO O DELL'ABSIDE



19 – ELEMENTI DI COPERTURA: AULA



20 - ELEMENTI DI COPERTURA: TRANSETTO



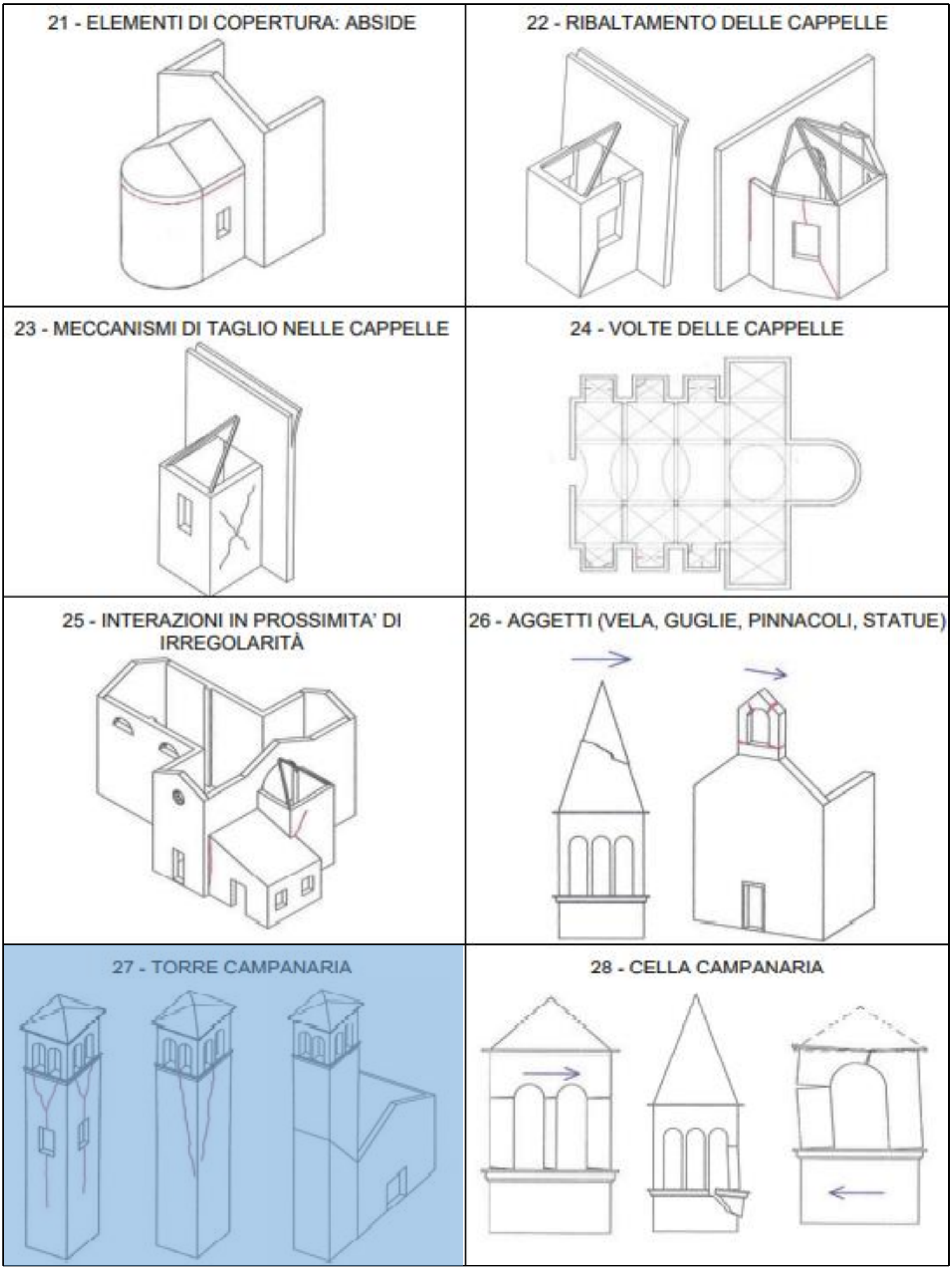


Figure 16 - Damage Mechanism

2.4 Model A-DC

As already mentioned in the previous section, the "Model A-DC" form is used for the assessment of damage in the cultural heritage of masonry churches, considering all the mechanisms previously introduced. It is worth recalling that this model of A-DC board is valid only for masonry constructions, and not for other types, for example, if you consider concrete constructions, you will have to take as a reference the 1st level AeDES card.

This was not the first usable model of survey form, as already in 1987, following several seismic episodes, the first form useful for the detection of damage was proposed, which took the name of GNDT – Model S3. *"At the end of the activities, the data sheets for the detection of damage to movable and immovable property belonging to the national cultural heritage were approved by Interministerial Decree of 3 May 2001, published in the Official Gazette of 21 May 2001 n. 116.* "[3]

In this form approved in 2001, the damage mechanisms present were only 18, subsequently then through the *Decree of the President of the Council of Ministers of 23 February 2006,* 10 new damage mechanisms were added, thus reaching the 28 that we have already considered and showed previously.

The form is divided into two sections, in turn divided into different fields.

- This first section consists of 13 subsections called "A" and expresses the general information for the proper conduct of the survey.
- The second section is also divided into 14 subsections, from 14 to 27, which provide information on maintenance and damage assessment, the survey, and any measures to be taken in the event of a risk. In this section, we will find in point A_{16} the damage mechanisms that have been listed above.

The A-DC form reporting the 28 mechanism is used in damage assessment, when the earthquake has occurred; yet, the reference to the 28 mechanism may be used, in a preventive approach, for a vulnerability assessment, that is, to identify mechanism that may develop in case of an earthquake; the mechanism may be more or less likely to develop depending on the presence of factors like cracks or, in the opposite, be limited by tie-rods and other factors.

[3] Presidency of the Council of Ministers – Department of Civil Protection – "Manual for the compilation of the form for the survey of damage to cultural heritage, Churches MODEL A – DC"

2.4.1 Damage mechanism of the case study

Knowledge of damage mechanisms thus makes it possible to examine the seismic behavior of churches in a safer and more in-depth way. Obviously, among all the mechanisms mentioned, only a few are recognizable within the structure we analyzed. In particular, the most usual mechanisms concern:

- *The façade tilting mechanism,* one of the most frequent and recurrent mechanisms in the event of an earthquake; it consists of the rigid rotation of the wall considered around a horizontal hinge placed on the ground. For the façade there are also other sub-mechanisms that depend on the position of formation of the hinge around which the rotation takes place.

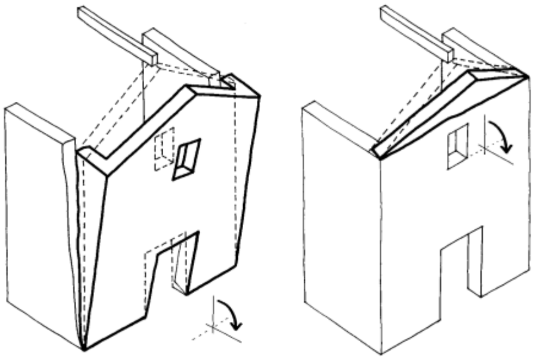


Figure 17 - Cracking mechanism of the façade

Usually, this type of damage occurs in the absence of homogeneity of the masonry and binding with the bracing or angled. The presence of an internal tie-rod, shown in the previous chapter, is useful to try to prevent this mechanism.

- *The out of plane rotation of the side walls, the* behavior of these parts of the structure also depends on other elements, such as the possible connections with other building elements such as, in our case, the sacristy, the chapels and the roof.

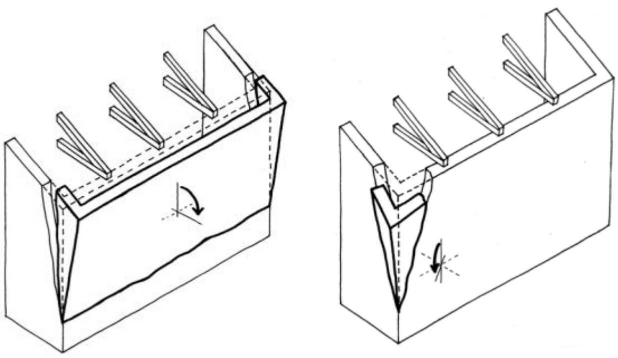


Figure 18 - Side wall cracking mechanism

- *The mechanisms of damage to the bell tower,* this, after the façade, is one of the most subject and vulnerable elements. For the bell tower the mechanisms are related to both the tower and the belfry, as shown.

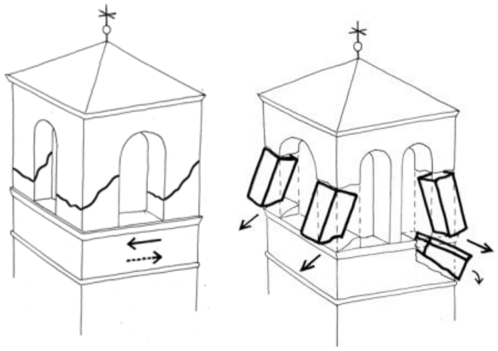


Figure 19 - Meccanismo di fessurazione della cella campanaria

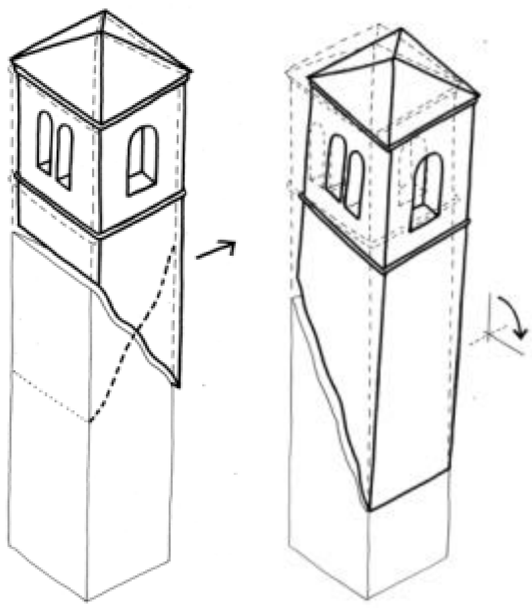


Figure 20 - Meccanismi di fessurazione della torre campanaria

2.5 Characteristics of the model for analysis

In order to set up a model for seismic analyses, different types of data and information are necessary. The NTC code indicates a general list of requirements that the engineer needs to adapt to the context.

Among these we find:

- Historical-critical analysis, to define the history and the changes undergone over time by the building itself.
- The survey must be complete, considering the previous point, therefore any changes suffered over time.
- Properties and characteristics of materials, through known information or any instrumental investigations in situ.
- Levels of knowledge and confidence, is the parameter at the base of the analysis of a building, useful for evaluating the mechanical properties of the structure:
    - LC1, survey of the artifact, limited checks, with FC[4] = 1.35; This level of knowledge is related to limited in situ checks on construction details, where only the materials in place are analyzed.
    - LC2, survey of the artifact, extended checks, with FC = 1.2, construction details and investigations on materials are analyzed.
    - LC3, extensive checks and in-depth investigations, FC = 1, geometric surveying, extensive checks on details and in-depth analysis are carried out.

  The definition of a confidence factor, depending on the amount of information attained, is important for the assessment of seismic safety and the planning of measures to improve cultural assets.

- Actions, calculation of all load combinations, carefully studying structural, non-structural permanent loads and variable loads.
- Design the seismic improvement intervention while considering how vulnerable the situation is., depending on the understanding of the building.

[4] Confidence Factor FC, the definition will be given in the next paragraph

Livello di Conoscenza	Geometria	Dettagli costruttivi	Proprietà dei materiali	Metodi di analisi	FC
LC1		verifiche in situ limitate	Indagini in situ limitate Resistenza: valore minimo di Tabella C8A.2.1 Modulo elastico: valore medio intervallo di Tabella C8A.2.1		1.35
LC2			Indagini in situ estese Resistenza: valore medio intervallo di Tabella C8A.2.1 Modulo elastico: media delle prove o valore medio intervallo di Tabella C8A.2.1		1.20
LC3	Rilievo muratura, volte, solai, scale. Individuazione carichi gravanti su ogni elemento di parete Individuazione tipologia fondazioni. Rilievo eventuale quadro fessurativo e deformativo.	verifiche in situ estese ed esaustive	Indagini in situ esaustive -caso a) (disponibili 3 o più valori sperimentali di resistenza) Resistenza: media dei risultati delle prove Modulo elastico: media delle prove o valore medio intervallo di Tabella C8A.2.1 -caso b) (disponibili 2 valori sperimentali di resistenza) Resistenza: se valore medio sperimentale compreso in intervallo di Tabella C8A.2.1, valore medio dell'intervallo di Tabella C8A.2.1; se valore medio sperimentale maggiore di estremo superiore intervallo, quest'ultimo; se valore medio sperimentale inferiore al minimo dell'intervallo, valore medio sperimentale. Modulo elastico: come LC3 – caso a). -caso c) (disponibile 1 valore sperimentale di resistenza) Resistenza: se valore sperimentale compreso in intervallo di Tabella C8A.2.1, oppure superiore, valore medio dell'intervallo; se valore sperimentale inferiore al minimo dell'intervallo, valore sperimentale. Modulo elastico: come LC3 – caso a).	Tutti	1.00

Figure 21 - Levels of knowledge and confidence

Tipologia di muratura	Malta buona	Giunti sottili (<10 mm)	Ricorsi o listature	Connessione trasversale	Nucleo scadente e/o ampio	Iniezione di miscele leganti	Intonaco armato *
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	1,5	-	1,3	1,5	0,9	2	2,5
Muratura a conci sbozzati, con paramen-to di limitato spessore e	1,4	1,2	1,2	1,5	0,8	1,7	2
Muratura in pietre a spacco con buona tessitura	1,3	-	1,1	1,3	0,8	1,5	1,5
Muratura a conci di pietra tenera (tufo, calcarenite, ecc.)	1,5	1,5	-	1,5	0,9	1,7	2
Muratura a blocchi lapidei squadrati	1,2	1,2	-	1,2	0,7	1,2	1,2
Muratura in mattoni pieni e malta di calce	1,5	1,5	-	1,3	0,7	1,5	1,5

* Valori da ridurre convenientemente nel caso di pareti di notevole spessore (p.es. > 70 cm).

Figure 22 - Parameters for different typology of masonry

2.6 Confidence Factor for a masonry building

A confidence factor "FC" must be defined, the value of which is between 1 and 1.35, depending on knowledge level as seen before; the use of this factor varies depending on the model being considered.

1. Models that consider the limit state: in this case the factor is applied directly to the capacity of the structure, reducing acceleration. In this instance, when the model of a rigid body is considered and the material's strength is disregarded, the confidence factor directly affects the structure's capacity, lowering the acceleration associated with various limit states.
2. Models that consider the deformations, in this case the confidence factor is applied to the properties of the materials, in order to reduce the resistance to be considered.

Generally, the confidence factor for masonry buildings of cultural heritage level is calculated as follows, as defined in the PcdM:[5]

$$FC = 1 + \sum_{k=1}^{4} F_{Ck}$$

where we define F_{Ck} a partial confidence factor that varies (1,4) according to the levels of depth of investigation, as reported in the table below.

Table 3 – Guide Lines for Cultural Heritage Tab 4.1 - Chapter 4.2

Rilievo geometrico	rilievo geometrico completo	$F_{C1} = 0.05$
	rilievo geometrico completo, con restituzione grafica dei quadri fessurativi e deformativi	$F_{C1} = 0$
Identificazione delle specificità storiche e costruttive della fabbrica	restituzione ipotetica delle fasi costruttive basata su un limitato rilievo materico e degli elementi costruttivi associato alla comprensione delle vicende di trasformazione (indagini documentarie e tematiche)	$F_{C2} = 0.12$
	restituzione parziale delle fasi costruttive e interpretazione del comportamento strutturale fondate su: a) limitato rilievo materico e degli elementi costruttivi associato alla comprensione e alla verifica delle vicende di trasformazione (indagini documentarie e tematiche, verifica diagnostica delle ipotesi storiografiche); b) esteso rilievo materico e degli elementi costruttivi associato alla comprensione delle vicende di trasformazione (indagini documentarie e tematiche)	$F_{C2} = 0.06$
	restituzione completa delle fasi costruttive e interpretazione del comportamento strutturale fondate su un esaustivo rilievo materico e degli elementi costruttivi associato alla comprensione delle vicende di trasformazione (indagini documentarie e tematiche, eventuali indagini diagnostiche)	$F_{C2} = 0$
Proprietà meccaniche dei materiali	parametri meccanici desunti da dati già disponibili	$F_{C3} = 0.12$
	limitate indagini sui parametri meccanici dei materiali	$F_{C3} = 0.06$
	estese indagini sui parametri meccanici dei materiali	$F_{C3} = 0$
Terreno e fondazioni	limitate indagini sul terreno e le fondazioni, in assenza di dati geotecnici e disponibilità d'informazioni sulle fondazioni	$F_{C4} = 0.06$
	disponibilità di dati geotecnici e sulle strutture fondazionali; limitate indagini sul terreno e le fondazioni	$F_{C4} = 0.03$
	estese o esaustive indagini sul terreno e le fondazioni	$F_{C4} = 0$

[5] PcdM Directive 9 February 2011 (1)

2.7 Non-destructive tests for the masonry

One of the most widely used methods for studying the behavior and characteristics of masonry is the technique of non-destructive testing, NDT also known as non-destructive examination (NDE), non-destructive inspection (NDI) and non-destructive evaluation.

With this type of test, it is possible to analyze the structure under consideration without damaging the structure itself; sometimes, only the surface is affected, but without affecting the integrity of the area.

From this type of test is very easy to evaluate some features of the masonry[6], like:

- Variations in wall construction.
- Missing wall insulation.
- Subsurface anomalies such as voids, near-surface cracks.
- Thermal bridging of mortar obstructions in wall drainage cavities.
- Definition of the material characteristic.
- Evaluation of the Elastic Modulus.
- Evaluation of the thickness of the structural elements.

[6] Michael P. Schuller, P.E - "Non-destructive testing and damage assessment of masonry structures"

3 IN SITU TESTS

For the church of Villavesco, a campaign of non-destructive diagnostic tests has been organized and conducted. This chapter will describe the elaboration of the images of the tests; the tests were carried out in collaboration with Irene Grave, who described them in detail in her thesis [1], and with whom I personally collaborated in carrying out the tests.

The tests carried out are generally non-destructive and concern the analysis of the structure to obtain significant information of the structural elements. Thermographic, sonic and thickness measurements were carried out.

The following is the map showing the location of all the tests carried out on the Church under analysis.

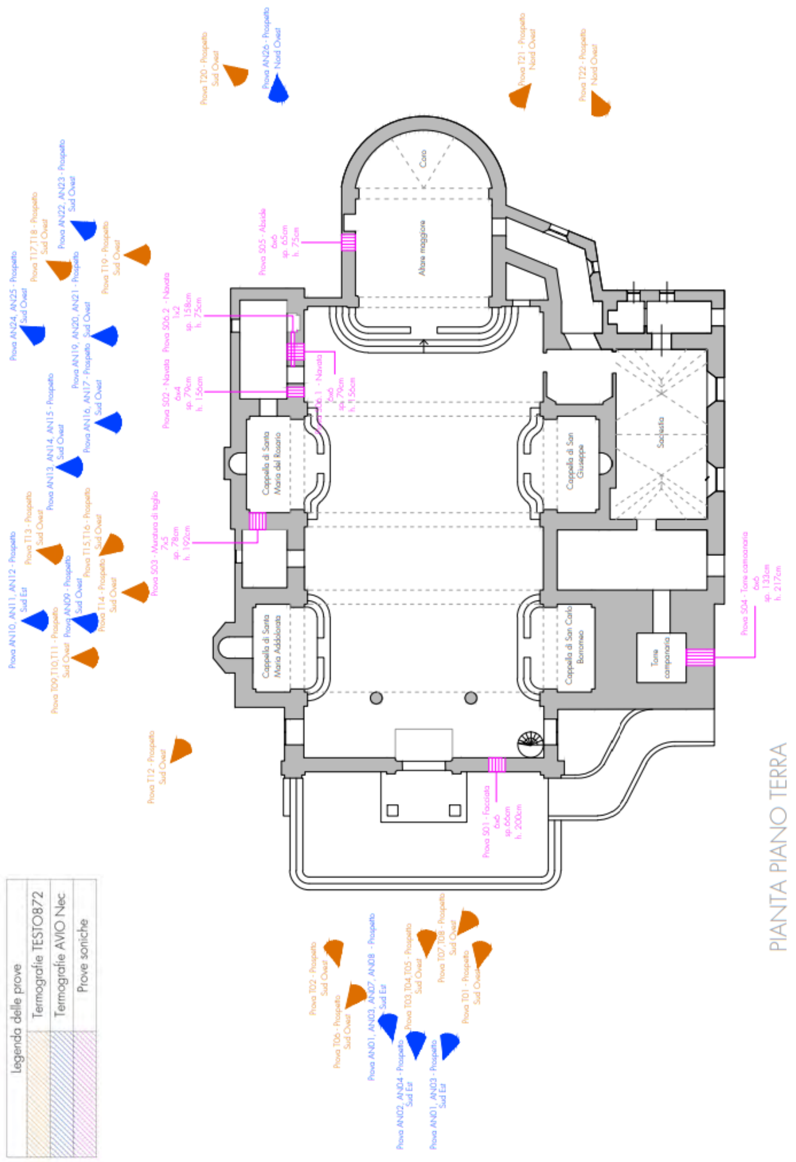


Figure 23 - Location of the tests carried out

3.1. Thermography

This type of non-destructive test is treated by the UNI 9252: 1988 standard, it is widely used as it is very advantageous because it allows to identify the composition (morphology) of the plastered walls, where it is not possible or in any case you want to avoid the use of destructive tests. Thermography is a type of non-destructive analysis that is based on the acquisition of infrared images, it was developed in the 1960's by the military.

This process is very effective as it considers the heating and cooling capacity of the walls thanks to evaporation; Thermography, being a diagnostic tool that reports images, allows to obtain a mapping of the studied area subject to evaporation and therefore evaluate where the flow is greater. The use of this test occurs when the bodies are subjected to thermal stress, which affects the thermal conductivity and the specific heat of the material under analysis; specific heat expresses the ability of a material to retain heat; thus, the higher the thermal conductivity, the faster its heating will be and vice versa. The resulting heat flux will be a function of the thermal conductivity, density, and specific heat of the material; If the wall apparatus is composed of several materials, you will notice temperature differences, visible from the infrared image.

Thermography can be done passively or actively:

1. In the passive way you have only one room temperature and natural.
2. In the active method, on the other hand, we proceed with the heating of the surface involved in the analysis (not considered in this thesis).

This type of proof is based on the Stefan-Bolzmann law, which states that:

"*Every object at temperature T emits an amount of energy proportional to the fourth power of its absolute temperature*"

$$E = e \cdot \sigma \cdot T^4 \cdot d$$

Where:

- T is the absolute temperature [K].
- E is the energy flux [W/m^2].
- σ is a constant 5,67 · 10^{-8} [W/m^{2}K^4].
- e is the emissivity.

To have a better evaluation and having an accurate knowledge of both the characteristics and geometry of the structure under study, some fundamental tests were carried out in most of the church environments to get more information.

The analyses were carried out using the TESTO 872 thermal imaging camera; this object is useful in numerous fields, for example, industrial maintenance or building diseases such as thermal bridges. In the first inspection, these tests were carried out in non-optimal weather conditions, as the weather was cloudy and rainy; the South-West façade is the one that has been more subject to sunlight, so it has shown better behavior.

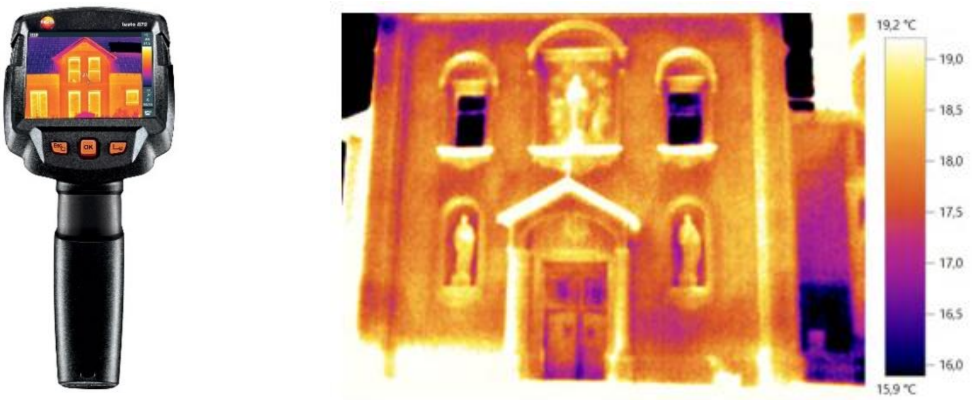


Figure 24 – Instrument used and first thermography results

To calibrate the thermograph, it starts with the acquisition of environmental data, such as temperature, humidity, and dew point. As a basic image we report the first shot.

This methodology is very useful in the building sector because it can find a lot of realization, like detachment of plaster, presence of humidity.

Below are some images relating to the most significant thermographic tests; all the images reported were shared with the Thesis of Irene Grave [1].

3.1.1. Results

- **Test T07**

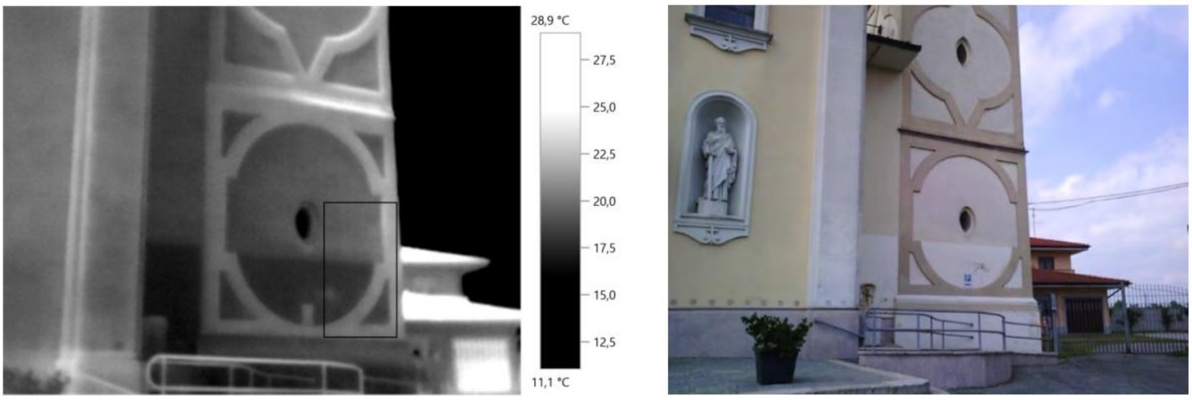


Figure 25 – Test T07, South East side and Bell Tower

From the image on the left above it can be seen that the base of the bell tower is composed of a different material; at this point it will then highlight a greater thickness of the masonry and of the cover, while above this point the masonry is visible. The different coloration represents two different temperatures, and they have two different thermal inertias.

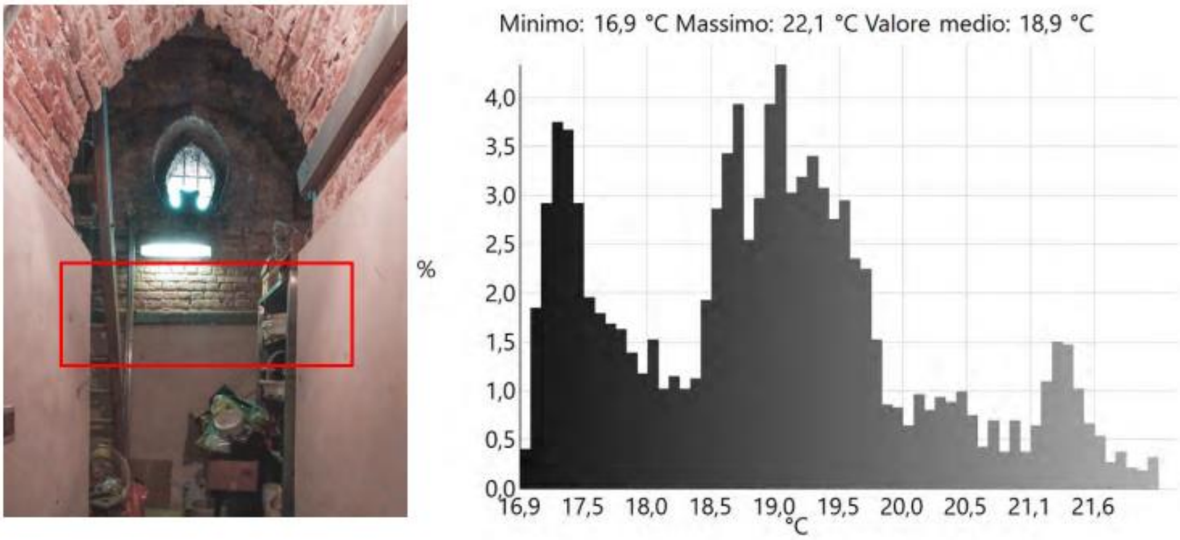


Figure 26 – Difference of materials inside the bell tower and thermography

As can be seen from the thermographic trend, the values are not constant but highly variable, principally 17.5° and 21.6°; this is probably due to the use of different materials.

- **Test T09**

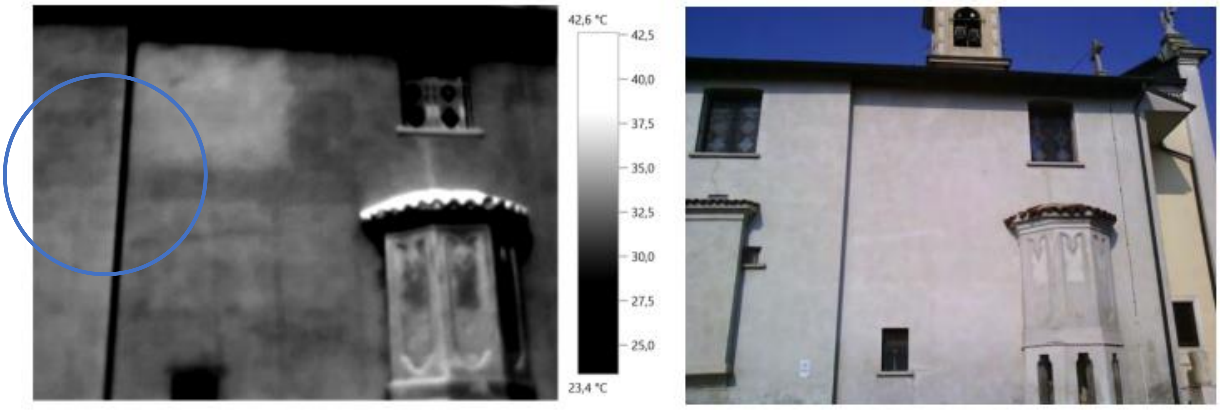
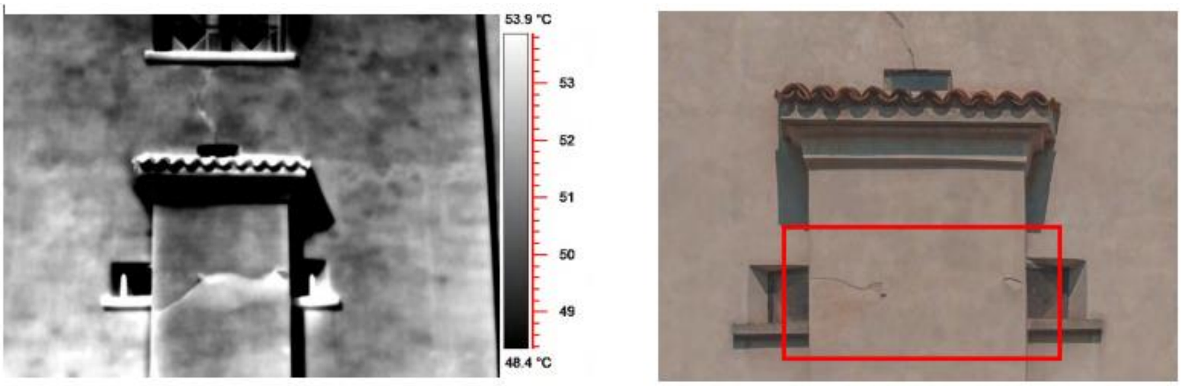


Figure 27 - Test T09, Thermographic images, Southwest elevation

In this image you can see a great discontinuity in the left side, the two elements are composed of different materials, in fact the temperature in that zone is very higher respect to the other; the one with the higher temperature is more porous, as it absorbs heat more easily with respect to the darker one. In these two images you can also see a large crack starting at the base of the window of the upper; here the coloration is lighter because, given the presence of air, the temperature is higher.

- **Prova AN14**



The ashlars of the masonry are not always regular, as the temperature inside it is variable. A thermal discontinuity is clearly visible.

3.2. Sonic and ultrasonic test

To evaluate the overall behavior of the structure, sonic tests were performed in different points of the church. Sonic tests are very important for the characterization of the structural elements and, not to be taken for granted, they are non-destructive tests and therefore return qualitative results. These tests are based on the transmission of elastic waves that, having a given speed, suggest the type of material of the element studied and possibly the presence of some defect or some mortar joint, which usually dampens the speed of the wave. These tests can be carried out according to three test methods:

- For transparency, the emitter and the receiver are on opposite sides
- For radial transmission, the two instruments are on adjacent sides
- On the surface, the least suitable for masonry structures

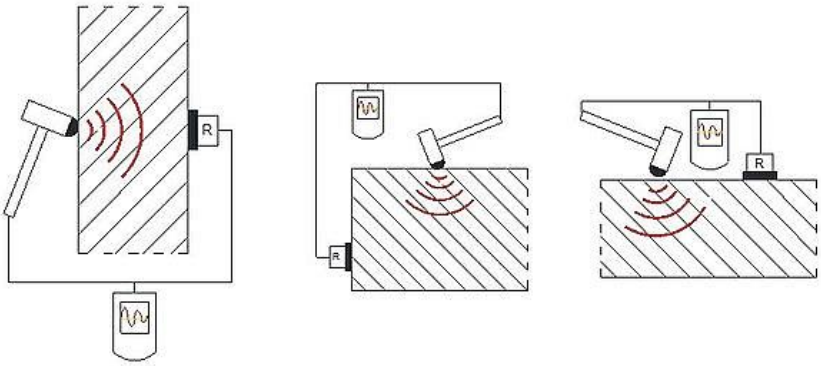


Figure 28 – Scheme of sonic test modalities

In our case study, it was chosen to perform a test in transparency, where the receiving probe and the emitting one are positioned on the two different sides of the structure.

The test therefore consists in evaluating the time of ethers between the emitter and the receiver, and, knowing then the trajectory of the wave, then the thickness of the walls, it is possible to trace the speed data of the wave that suggests the state of conservation or the type of material, according to this formula:

$$V = \sqrt{\frac{E}{\rho}}$$

where "E" is the Young Modulus" and "ρ" is the density of the material.

For this type of test it is necessary to:

- Prepare a 36-point grid (usually, then depending on the case in question), on 1 square meter of surface, to have a sufficiently representative sample of the structure, with 15cm each other.
- Use an instrumented hammer and lightly beat it on each point of the grid.
- Detect the intensity of the blow by means of a probe.
- Evaluate the speed of propagation.

Investigations were carried out with the following tools:

- The transmitter generates the impulse
- The receiver
- The wireless, acquisition with the data

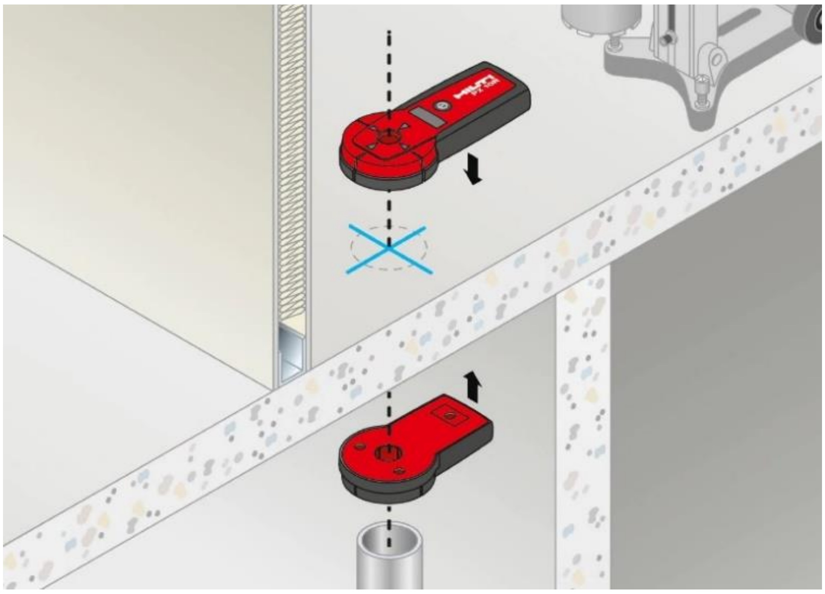


Figure 29 - Illustration of the methodology

3.2.1. Wall thickness measurement

Given the date of construction of the church, and the possible variability of the thickness of the walls, which is assumed to be very high, measurements have been performed on the main load-bearing walls, to make the three-dimensional model for seismic analysis more reliable. This instrument is used to measure the thickness of the wall and ceiling for a simplified choice of the correct drill bit or crown and to determine the outlet point of the drill bit and transfer drilling positions when working through concrete and masonry walls and ceilings. Hilti PX 10 Transpointer helps to drill safely, reducing the risk of hitting a pipe, cable or recovery iron. The PX 10 T transmitter generates a magnetic field that penetrates bricks, wood, concrete, and reinforced concrete to allow alignment with the PX 10 R receiver.

The main characteristics of this instruments are:

- It allows you to perform drilling operations quickly and efficiently.
- It allows you to freely choose the direction of drilling, for example when drilling through slabs or before complex coring applications.
- Drilling operations can also be marked in multi-level slabs or non-aligned partitions.
- It prevents damage caused by drilling through insulation or coatings ensuring that it is performed in the correct direction.

Figure 30 - Instrument for the wall thickness measurement

To make these measurements, the above instrument was used; the two objects are arranged on two different sides of the wall, in the same position as illustrated in Figure and, after finding the signal, the PX10R reports a range of values (very low) that represent the possible values of the thickness of the masonry; in our case, having relied on DWG outsourced, we verified that, what was reported was present in the range; in the event that the measurements were incorrect, the average value of the range was assumed.

The measurement was evident only when the two objects were perfectly superimposed; the arrows colored green, in fact, serve to indicate the exact position and are useful to follow to find the indicated point. Once the thickness of the chosen masonry is measured, the grid is composed to carry out the next necessary tests.

The following figure represents the measurement of the wall of the entrance door.

Figure 32 – Photographic representation during in situ testing

Figure 31 - Photographic representation during in situ testing

Once the thickness of the chosen wall has been measured, reference is made to the grid and the next step is to carry out subsequent measures.

3.2.2. Results of the sonic test

Some of the most important results to be considered in the following steps are listed here as from the [1] Irene Grave thesis. For each test the surface graph is reported, useful as it associates the area studied and affected by the grid with the speed at each point. This then produces a speed map that allows you to guess on average the type of materials and the condition of the structure.

Test S01

The first test is carried out on the façade; the velocity range in highly variable, between 400 and 1300 m/s, which indicated the presence of voids or materials not well connected, where velocity is higher the connection is better and so will be the material properties; viceversa, for lower velocities, materials properties are also lower.

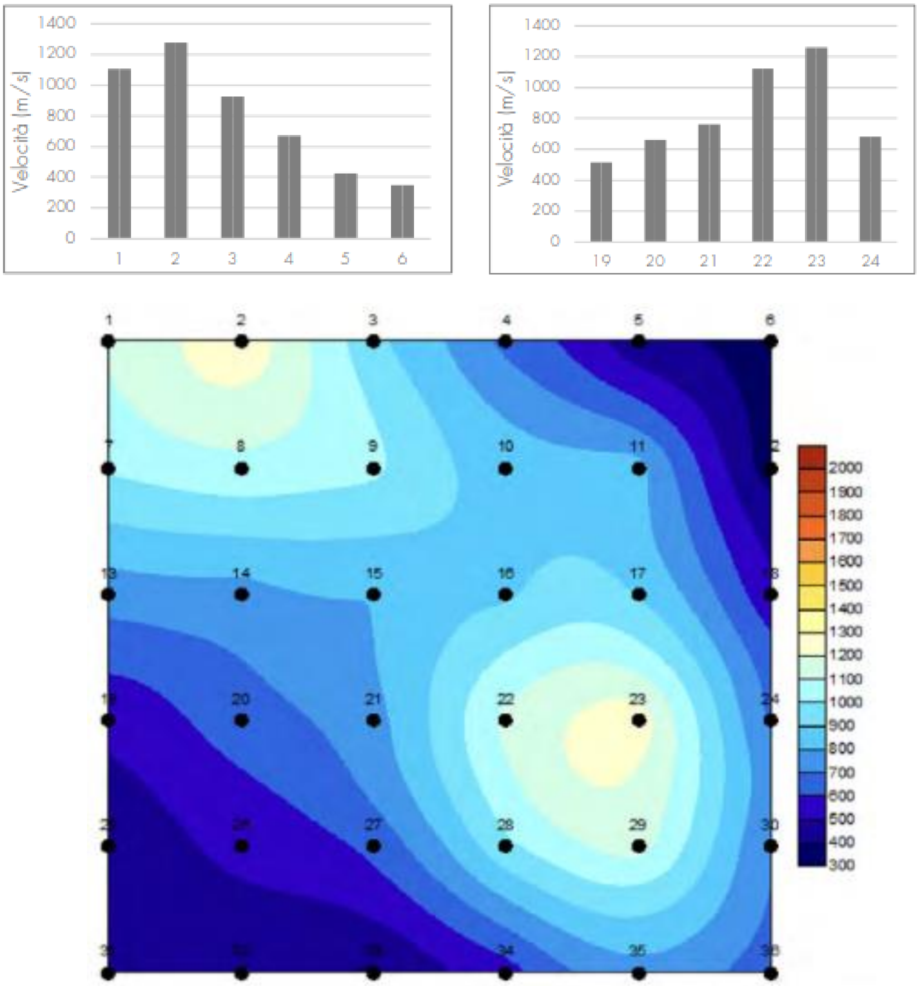


Figure 33 – Test S01, Facade

Generally, it can therefore be said that this part of masonry studied does not have a homogeneous behavior and the whole seems to be not very compact.

Test S04

Another of the most important tests concerns the bell tower, here the speeds are much higher than in the previous case, ranging from 100 m / s to about 1350 m / s.

Compared to the previous presentation, S01 relating to the façade, we can see that in this case the speed values are much higher; this is probably due to the subsequent construction of the bell tower in its current position and, therefore, to a use of better materials and methodologies than those used in the façade.

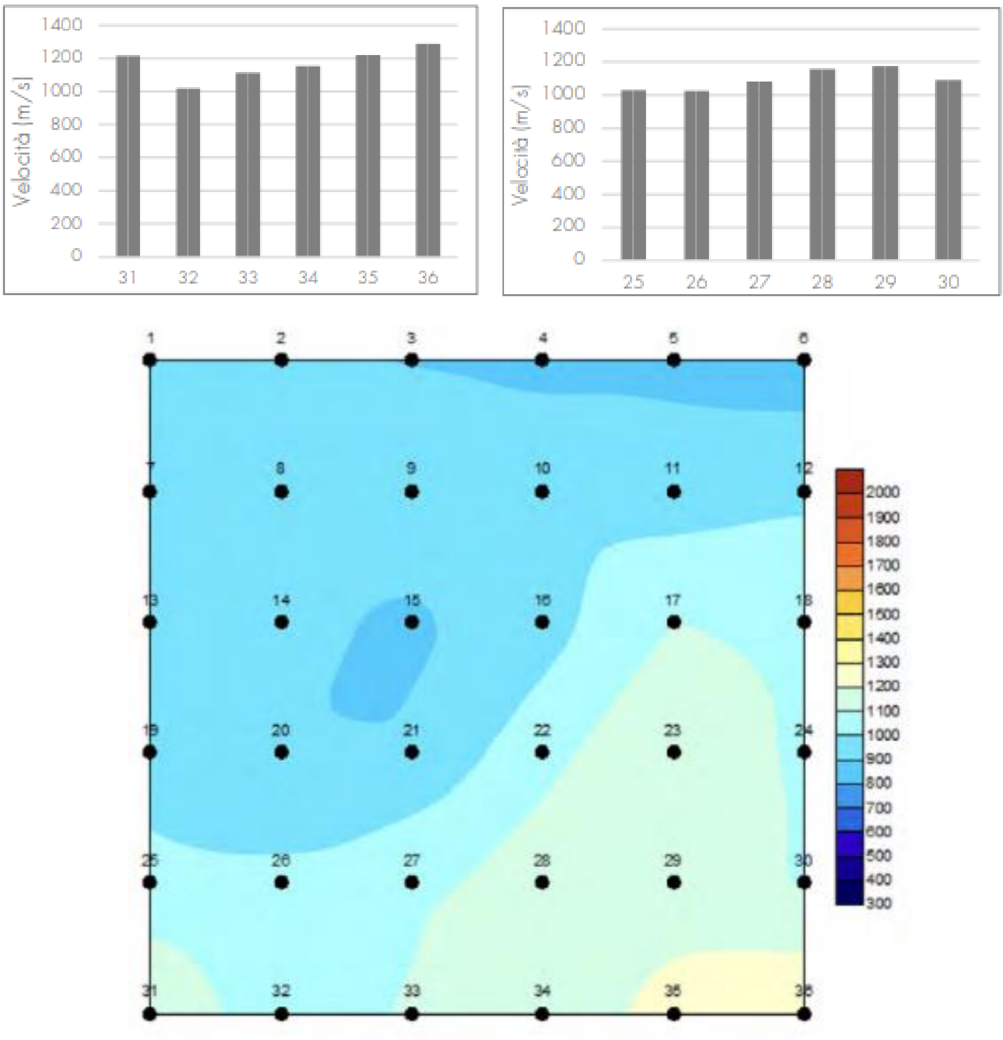


Figure 34 – Test S04, Bell Tower

The general condition of the bell tower looks quite good, even though the speeds do not give excellent values.

Test S05

One of the tests with worst results is from the apse, where speeds are very low, indicating high discontinuity in the materials. Despite of the presence of discontinuities and voids, speeds after all are constant in that grid, which implies that masonry was built in a regular way, even if the composition was by low quality.

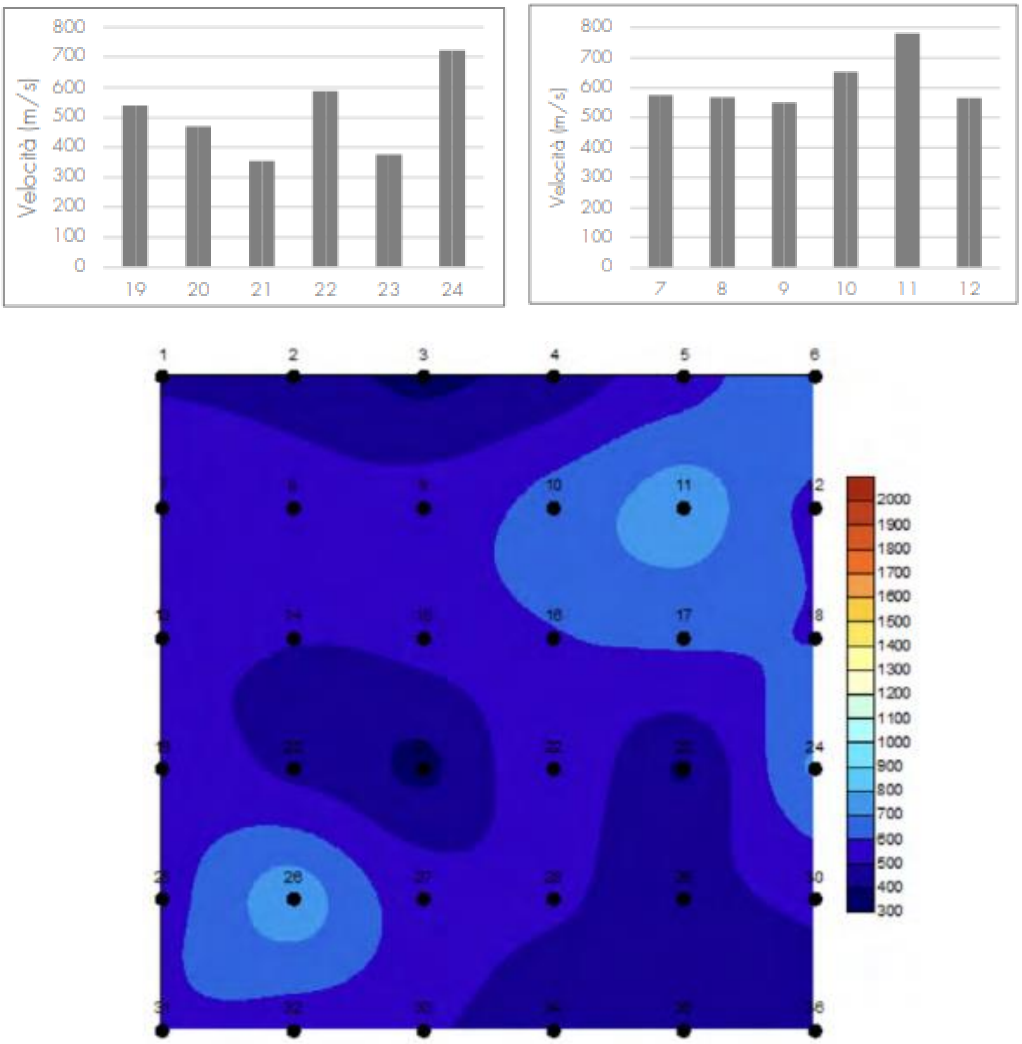


Figure 35 – S05 Test, Apse

From the tests carried out it can be assumed that the material by which the church is built is not very regular; the speeds are very uneven and therefore most likely, the materials used were many, due to continuous changes over time, and different from each other.

The current situation of the church in terms of maintenance and plaster is not optimal, probably also the blaster detachment causes the entry of air into the masonry and therefore causes discontinuity. Trying to make a comparison with the façade, it is likely that, from the results, the façade is composed of brick and mortar, while the nave, where the speed is much lower, is composed of a colder material, such as stone.

4 LOADS ANALYSIS

The loads are divided according to their intensity over time; according to the NTC 2018, there are different types of loads, listed below. As far as we need, we will consider only the permanent loads, due to the weight of the structure, and the seismic action, but for completeness all types of loads will be indicated and reported below.

1. Permanent (G):
    - Self-weight of the structural elements (G1).
    - Self-weight of all non-structural elements (G2).

2. Variable (Q): actions that act with instantaneous values that can be significantly different from each other during the nominal life of the structure:
    - Wind actions; *dynamic loads.*

3. **Seismic Action E**, which generally depends on the site's parameters, on the seismic hazard (probability that an earthquake can occur); there are horizontal and vertical components of the ground motion.

4.1 Evaluation of the seismic action "E"

According to "NTC – Technical Standards for Construction – 2018", seismic actions are defined starting from the seismic hazard of the construction site.

The seismic hazard is the probability that the ground will shake, considering different time scales. To analyze the seismic hazard, the probability of earthquakes must be predicted and the propagation of seismic waves with respect to the center of the earthquake must be estimated by means of a special law. The National Institute of Geophysics and Volcanology has represented a map of seismic hazard of the Italian National Territory, where the hazard itself is defined according to parameters of maximum expected horizontal acceleration to a_g, relative to a reference period V_R, a probability of exceedance P_{VR} and an elastic response spectrum. This map is the reference map for seismic hazard in Italy, with a probability of exceedance of 10% in 50 years.

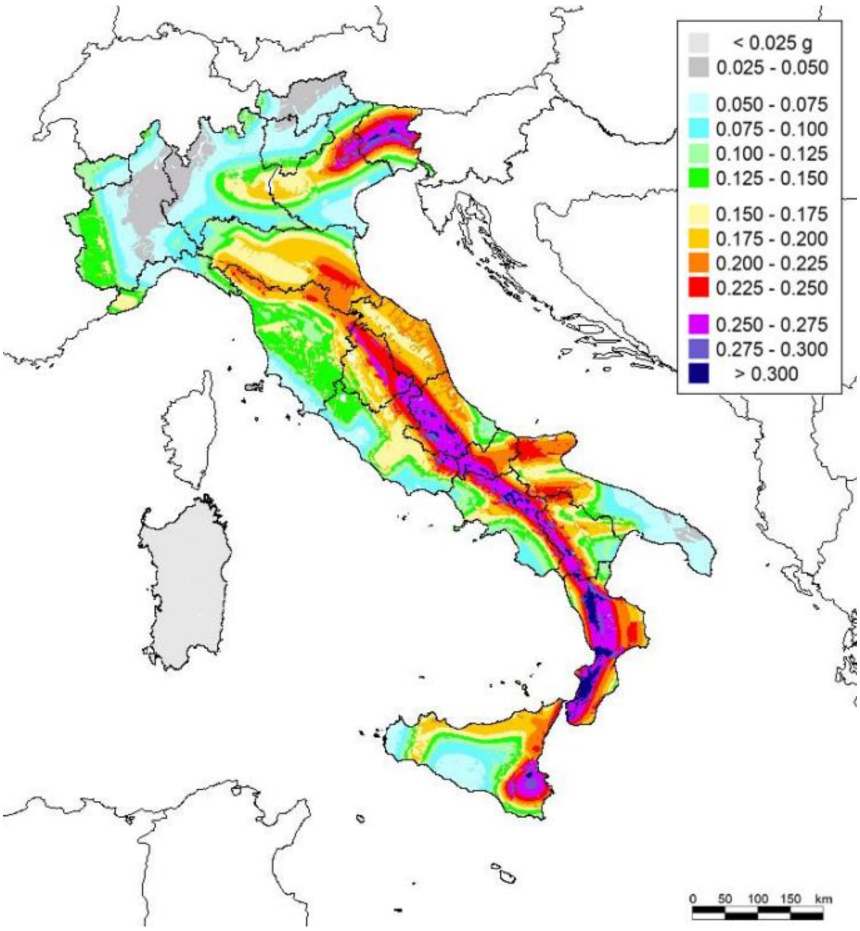


Figure 36 - Seismic hazard model MPS04-S1, INVG

As far as our case study is concerned, there are two maps available on the website of the National Institute of Geophysics and Volcanology, the first concerns ground acceleration, the second map instead the shaking on the ground.

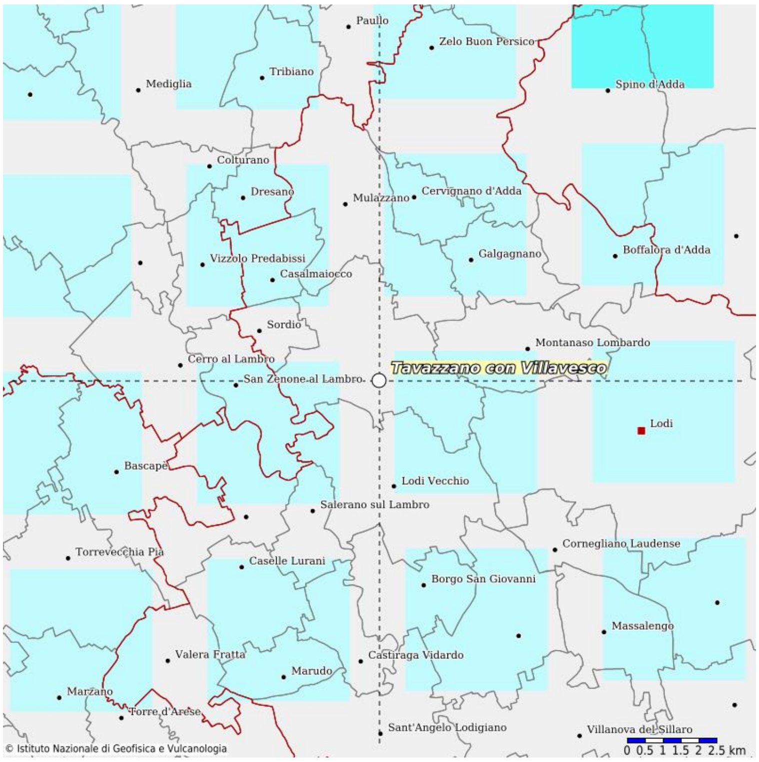


Figure 37 - Ground acceleration map for Tavazzano con Villavesco

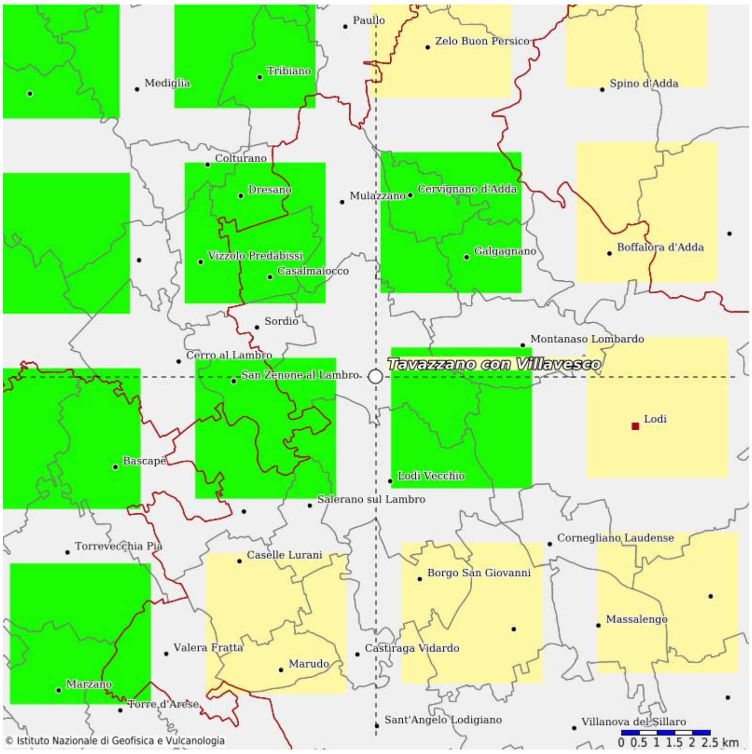


Figure 38 - Shaking map for case study in Tavazzano con Villavesco

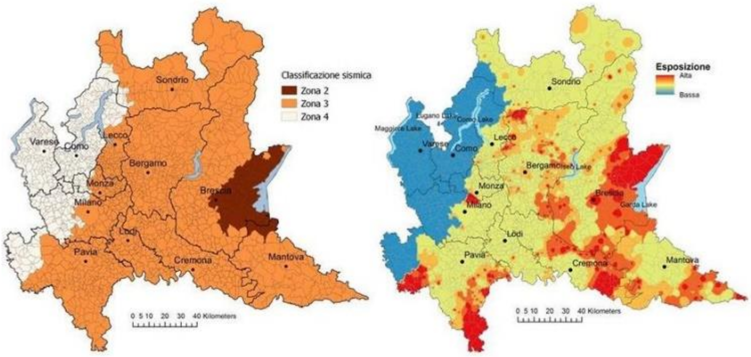
The values that are considered to define spectral shapes are:

- A_g maximum horizontal acceleration to the site.
- F_0, the maximum value of the spectrum amplification factor in horizontal acceleration.
- T_c, initial period of the section at constant speed of the spectrum in horizontal acceleration.

These parameters can be obtained from numerous software and programs that provide interactive maps of danger on the ground, after introducing the coordinates of the reference place. From the seismic point of view, Lombardia region is not very prone to dangerous earthquakes although, in recent years, several tremors have hit the region.

Seismic Zone	**Description**	**Acceleration with probability of exceeding 10% in 50 years [a_g]**	**Conventional maximum horizontal acceleration [a_g]**	**Number of municipalities with territories in the area (*)**
1	Indicates the most dangerous area, where strong earthquakes can occur.	$a_g > 0{,}25$ g	0,35 g	703
2	Area where strong earthquakes can occur.	$0.15\ g < a_g < 0.25\ g$	0.25 g	2224
3	**An area that may be subject to strong but rare earthquakes.**	**$0.05 < a_g < 0.15$**	**0.15 g**	**3002**
4	This is the least dangerous area, where earthquakes are rare, and the Regions have the power to prescribe the obligation of anti-seismic design.	$a_g \leq 0{,}05$ g	0.05 g	1982

The town of Tavazzano con Villavesco is in seismic zone 3, indicated in the Ordinance of the President of the Council of Ministers n. 3274/2003, updated with the Resolution of the Regional Council of Lombardy of 11 July 2014 n.2129 enforced on 10 April 2016.



4.1.1 Nominal Life, Class of Use and Reference Period

There are several steps to follow to obtain the values of seismic action to be applied to the structure considered, we start considering the "nominal life - it has been considered to be 50 years as normative requires":

Table 4 – Nominal Life according to the construction, Tab. 2.4.I NTC

Tipi di Costruzioni		**Valori minimi di V_N (years)**
1	Costruzioni temporanee e provisory	10
2	Costruzioni con livelli di prestazioni ordinari	50
3	**Costruzioni con livelli di prestazione elevati**	**100**

- Class 1, costruzioni con presenza solo occasionale di persone, edifici agricoli.
- Class 2, costruzioni il cui uso preveda normali affollamenti, senza contenuti pericolosi per l'ambiente e senza funzioni pubbliche e sociali essenziali. Industrie con attività non pericolose per l'ambiente. Ponti, opere infrastrutturali, reti viarie non ricadenti in Classe d'uso III o in Classe d'uso IV, reti ferroviarie la cui interruzione non provochi situazioni di emergenza. Dighe il cui collasso non provochi conseguenze rilevanti.
- Class 3, Costruzioni il cui uso preveda affollamenti significativi. Industrie con attività pericolose per l'ambiente. Reti viarie extraurbane non ricadenti in Classe d'uso IV. Ponti e reti ferroviarie la cui interruzione provochi situazioni di emergenza. Dighe rilevanti per le conseguenze di un loro eventuale collasso.
- Class 4, Costruzioni con funzioni pubbliche o strategiche importanti, anche con riferimento alla gestione della protezione civile in caso di calamità. Industrie con attività particolarmente pericolose per l'ambiente. Reti viarie di tipo A o B, di cui al DM 5/11/2001, n. 6792, "Norme funzionali e geometriche per la costruzione delle strade", e di tipo C quando appartenenti ad itinerari di collegamento tra capoluoghi di provincia non altresì serviti da strade di tipo A o B. Ponti e reti ferroviarie di importanza critica per il mantenimento delle vie di comunicazione, particolarmente dopo un evento sismico. Dighe connesse al funzionamento di acquedotti e a impianti di produzione di energia elettrica.

The "class of use" is then defined, coefficient of use C_U is defined, based on the class of use, as stated in table, according to the NTC 2018, equal in this case to Class II, 1.0.

Classe D'Uso	I	II	III	IV
C_U	0.7	**1**	1.5	2.0

Knowing this parameter, the "reference period" is then defined according to the table:

Table 5 – V_R values depending on V_N and C_U, Tab. C2.4.I - NTC

Vita Nominale V_N	Valori di V_R			
	Classe d'uso			
	I	II	III	IV
≤ 10	35	35	35	35
≥ 50	≥ 35	≥ 50	≥ 75	≥ 100
≥ 100	≥ 70	≥ 100	≥ 150	≥ 200

The probability (PVR): the probability is considered as the probability of exceedance in the reference period T_R, to which we must refer to identify the seismic action acting in each of the limit states considered and is reported in table 3.2.1 of NTC 2008.

Table 6 - Probability PVR depending on the limit state, Tab. 3.2.I - NTC

Stati Limite	P_{VR}: Probabilità di superamento nel periodo di riferimento V_R	
SLE	SLO	81%
	SLD	63%
SLU	**SLV**	**10%**
	SLC	5%

The "return period" of the structure is:

$$\boldsymbol{T_R} = \frac{-V_R}{\ln\left(1 - P_{V_R}\right)} = \frac{-50}{\ln(1 - 10\%)} = 475\ years$$

4.2 Design Spectrum

For the analysis of the structure, it a linear analysis method will be used, case for which we will need to find the behavior factor q to consider the effect of non-linear behavior. The behavior factor allows us to reduce the elastic spectrum and get the design spectrum.

The elastic response spectrum is a graph that is very useful as it provides for the periods of vibration (T), the value of the acceleration due to the earthquake, relative to the damping, and of course the parameters are related to the different types of soil.

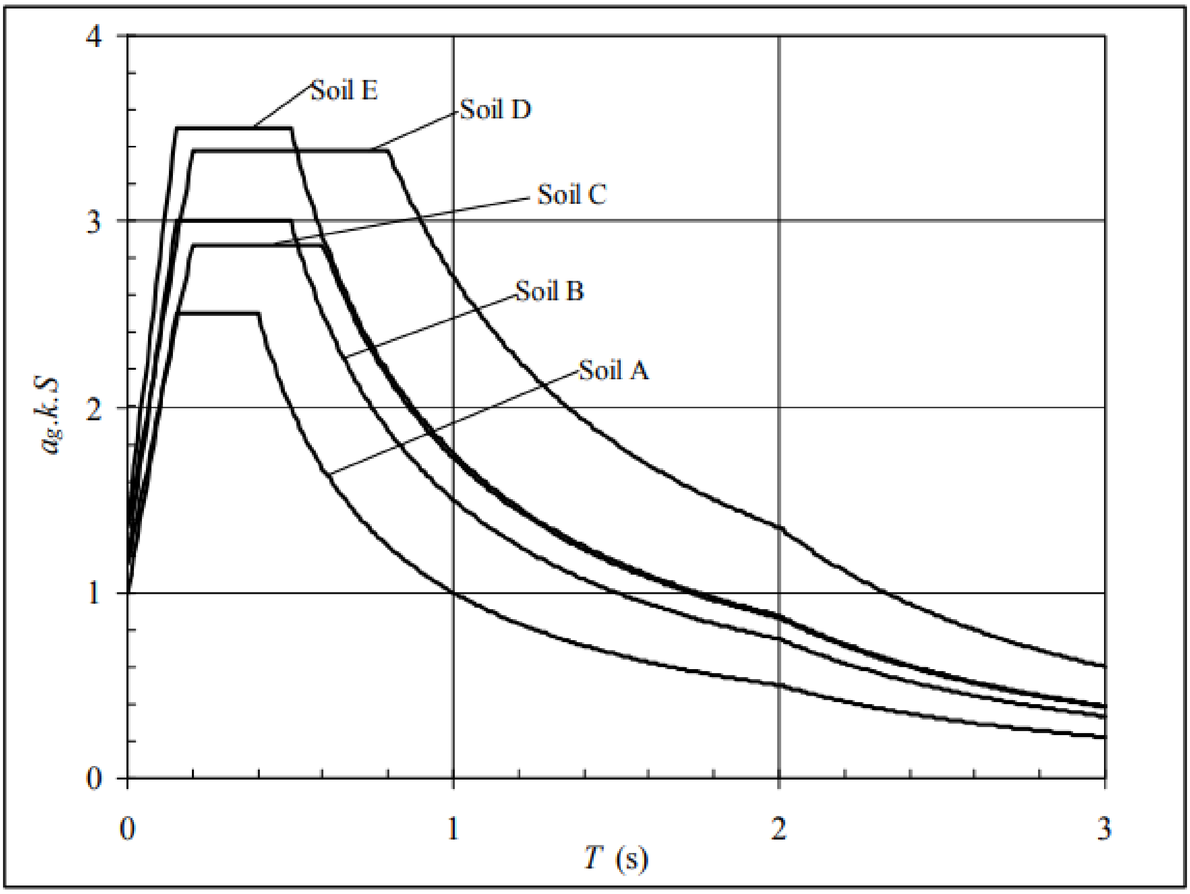


Figure 39 - Eurocode 8, Design Respose Spectrum for different types of soil

Once all the parameters have been defined, it's possible to obtain, for the specific site, in this case Tavazzano con Villavesco, the response spectrum.

4.2.1 Behaviour factor “q”

According to the code, when designing a new structure, the superior limit q_{lim} of the behavior factor relative to the SLV is calculated, using the following expression:

$$q_{lim} = q_0 \cdot K_R$$

- q0 is the base value of the behavior factor of the SLV.
- K_R is a factor depending on the regularity in height of the building, being equal to 1 for regular in height constructions and equal to 0,8 for non-regular in height constructions.

To obtain q_0 the Tab. 7.3.II of the NTC 2018is used according to the previously defined ductility class and the structural typology of the project.

Considering a dissipating structural behavior, we define the ductility class of the structure, which will be *Medium Ductility* (*CD”B”*).

Depending on the type of construction, there may be different values to be used in different linear analyses; the values taken are shown in the following table:

Costruzioni di muratura (§ 7.8.1.3)	
Costruzioni di muratura ordinaria	1,75 α_u/α_1
Costruzioni di muratura armata	2,5 α_u/α_1
Costruzioni di muratura armata con progettazione in capacità	3,0 α_u/α_1
Costruzioni di muratura confinata	2,0 α_u/α_1
Costruzioni di muratura confinata con progettazione in capacità	3,0 α_u/α_1

Figure 40 - Tab 7.2.II Maximum values of the base value q0 of the SLV behavior factor for different construction techniques and depending on the structural type and ductility class CD

For instance, in the case of a masonry building:

- Ordinary masonry, $\frac{\alpha_u}{\alpha_1} = 1.7$

Then it is possible to compute q_0:

$$q_0 = 1.75 \cdot \frac{\alpha_u}{\alpha_1} = 1.75 \cdot 1.7 = 2.975$$

The superior limit q_{lim} of the behaviour factor relative to the SLV is calculated, according NTC2018, using the following expression:

$$q_{lim} = q_0 \cdot K_R$$

$$q_{lim} = q_0 \cdot K_R = 2.975 \cdot 0.8 = 2.38$$

This is a value that shows that masonry can attain appreciable levels of post elastic behavior, ìf correctly designed. This value, however, is valid only for <u>newly designed</u> masonry buildings, for an existing building the data were not referred to post elastic behaviors.

It is reasonable to assume that every building may present some amount, even if small, of ductility, usually q =1.5 may be assumed. Here, however, reference will be initially made to the fully elastic spectrum, to define from it and from material strength the expected level of response.

4.2.2 Design Spectra, NTC-Excel

To obtain the spectrum, the NTC-Spectra Excel sheet is used, considering the nominal life and the class of use as previously calculated.

Categoria	Caratteristiche della superficie topografica
A	*Ammassi rocciosi affioranti o terreni molto rigidi* caratterizzati da valori di velocità delle onde di taglio superiori a 800 m/s, eventualmente comprendenti in superficie terreni di caratteristiche meccaniche più scadenti con spessore massimo pari a 3 m.
B	*Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti*, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 360 m/s e 800 m/s.
C	*Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti* con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 180 m/s e 360 m/s.
D	*Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti*, con profondità del substrato superiori a 30 m, caratterizzati da un miglioramento delle proprietà meccaniche con la profondità e da valori di velocità equivalente compresi tra 100 e 180 m/s.
E	*Terreni con caratteristiche e valori di velocità equivalente riconducibili a quelle definite per le categorie C o D*, con profondità del substrato non superiore a 30 m.

Figure 41 - Tab. 3.2.II – Categorie di sottosuolo che permettono l'utilizzo dell'approccio semplificato.

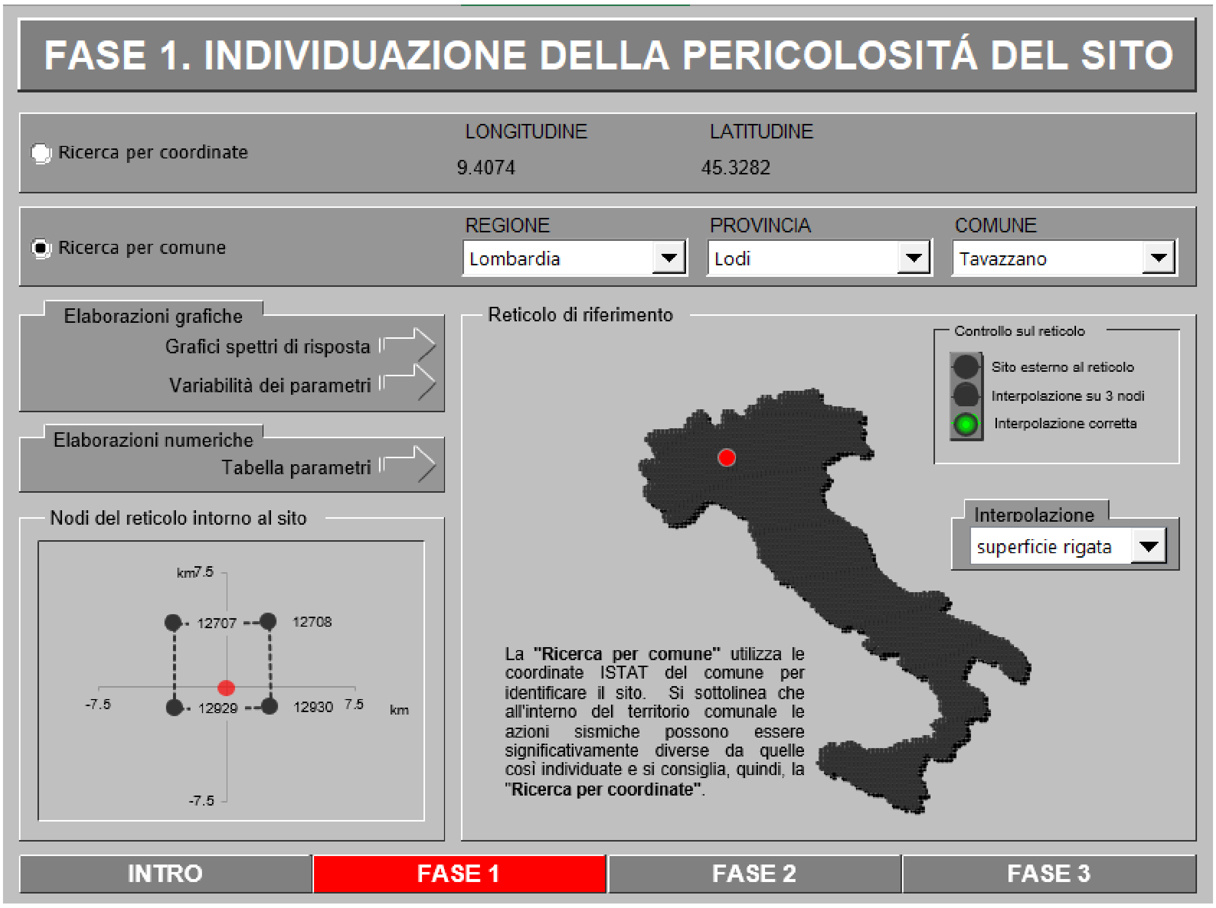
Categoria	Caratteristiche della superficie topografica
T1	Superficie pianeggiante, pendii e rilievi isolati con inclinazione media i ≤ 15°
T2	Pendii con inclinazione media i > 15°
T3	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media 15° ≤ i ≤ 30°
T4	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media i > 30°

Figure 42 - Tab. 3.2.III – Categorie topografiche

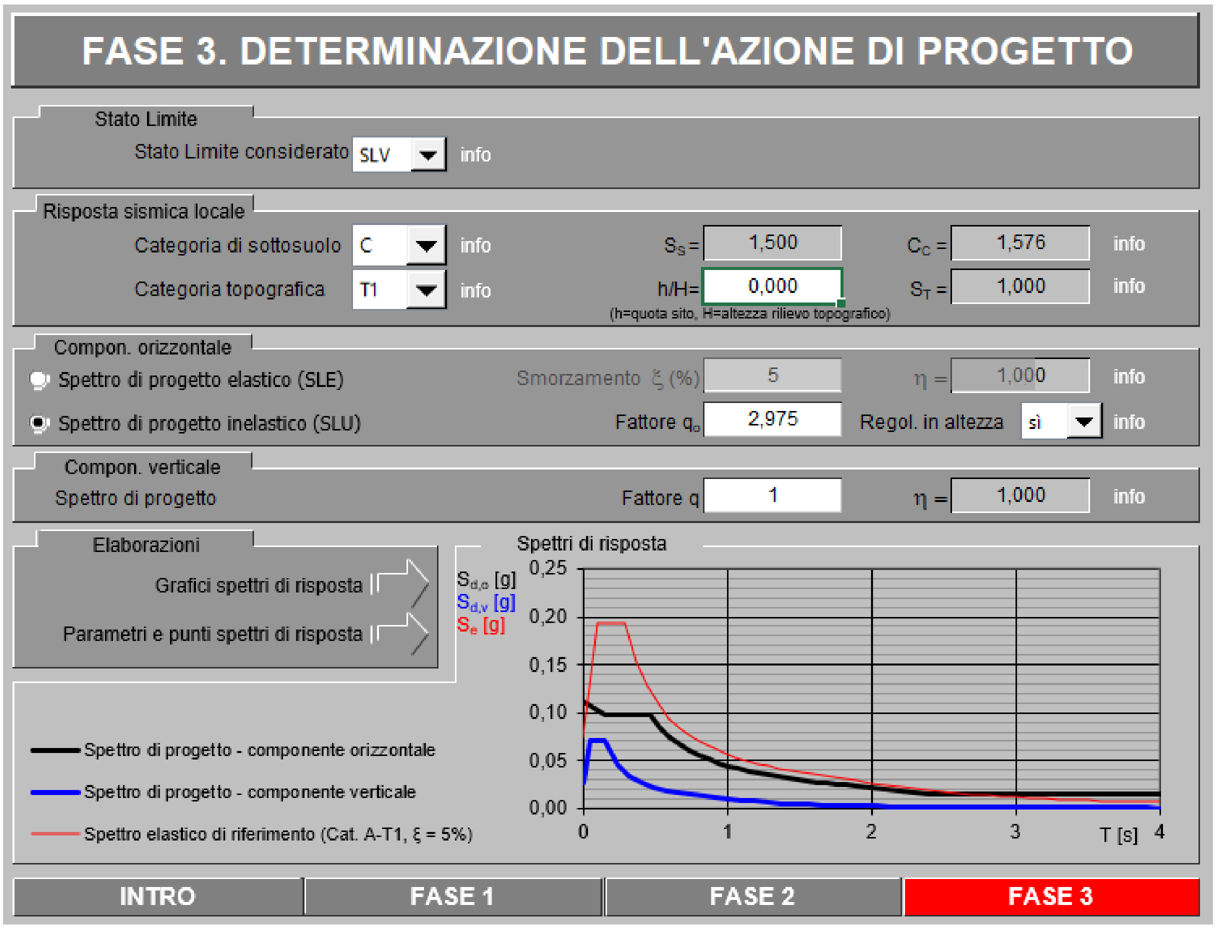
For the identification of the fundamental parameters, it is necessary to resort to the use of the Excel NTC-Spectra, which provides the response spectra representative of the components (horizontal and vertical) of the seismic project actions for the generic site of the national territory; this file is usually available from the site of the **Superior Council of Public Works**, the site is currently under modification and updating.

To use this file it is necessary, as you will see from the following pages, to enter some of the nominal values calculated above.


INTRO
D.M. 14 gennaio 2008 - Approvazione delle Nuove Norme Tecniche per le Costruzioni
Spettri di risposta ver. 1.0.3
Il documento Excel SPETTRI-NTC fornisce gli spettri di risposta rappresentativi delle componenti (orizzontali e verticale) delle azioni sismiche di progetto per il generico sito del territorio nazionale.
La definizione degli spettri di risposta relativi ad uno Stato Limite è articolata in 3 fasi, ciascuna delle quali prevede la scelta dei valori di alcuni parametri da parte dell'utente:
FASE 1. Individuazione della pericolosità del sito (sulla base dei risultati del progetto S1 - INGV);
FASE 2. Scelta della strategia di progettazione;
FASE 3. Determinazione dell'azione di progetto.
La schermata relativa a ciascuna fase è suddivisa in sotto-schermate: l'utente può intervenire nelle sotto-schermate con sfondo grigio scuro mentre quelle con sfondo grigio chiaro consentono un immediato controllo grafico delle scelte effettuate. In ogni singola fase l'utente può visualizzare e stampare i risultati delle elaborazioni -in forma sia grafica che numerica- nonchè i relativi riferimenti alle Nuove Norme Tecniche per le Costruzioni di cui al D.M. 14.01.2008 pubblicate nella G.U. n.29 del 04.02.2008 Suppl. Ord. n.30 e scaricabile dal sito www.cslp.it
Programma ottimizzato per una visualizzazione schermo 1024 x 768
La verifica dell'idoneità del programma, l'utilizzo dei risultati da esso ottenuti sono onere e responsabilità esclusiva dell'utente. Il Consiglio Superiore dei Lavori Pubblici non potrà essere ritenuto responsabile dei danni risultanti dall'utilizzo dello stesso.
INTRO
FASE 1
FASE 2
FASE 3


FASE 1. INDIVIDUAZIONE DELLA PERICOLOSITÁ DEL SITO
Ricerca per coordinate
LONGITUDINE
9.4074
LATITUDINE
45.3282
Ricerca per comune
REGIONE
Lombardia
PROVINCIA
Lodi
COMUNE
Tavazzano
Elaborazioni grafiche
Grafici spettri di risposta
Variabilità dei parametri
Elaborazioni numeriche
Tabella parametri
Nodi del reticolo intorno al sito
km 7.5
12707
12708
-7.5
12929
12930
7.5
km
-7.5
Reticolo di riferimento
Controllo sul reticolo
Sito esterno al reticolo
Interpolazione su 3 nodi
Interpolazione corretta
Interpolazione
superficie rigata
La "Ricerca per comune" utilizza le coordinate ISTAT del comune per identificare il sito. Si sottolinea che all'interno del territorio comunale le azioni sismiche possono essere significativamente diverse da quelle così individuate e si consiglia, quindi, la "Ricerca per coordinate".
INTRO
FASE 1
FASE 2
FASE 3


FASE 2. SCELTA DELLA STRATEGIA DI PROGETTAZIONE
Vita nominale della costruzione (in anni) - V_N
50
info
Coefficiente d'uso della costruzione - c_U
1,5
info
Valori di progetto
Periodo di riferimento per la costruzione (in anni) - V_R
75
info
Periodi di ritorno per la definizione dell'azione sismica (in anni) - T_R
info
Stati limite di esercizio - SLE
SLO - P_{VR} = 81%
45
SLD - P_{VR} = 63%
75
Stati limite ultimi - SLU
SLV - P_{VR} = 10%
712
SLC - P_{VR} = 5%
1462
Elaborazioni
Grafici parametri azione
Grafici spettri di risposta
Tabella parametri azione
Strategia di progettazione
T_R [anni]
10000
1000
100
10
45
75
712
1462
SLO
SLD
SLV
SLC
LEGENDA GRAFICO
Strategia per costruzioni ordinarie
Strategia scelta
INTRO
FASE 1
FASE 2
FASE 3

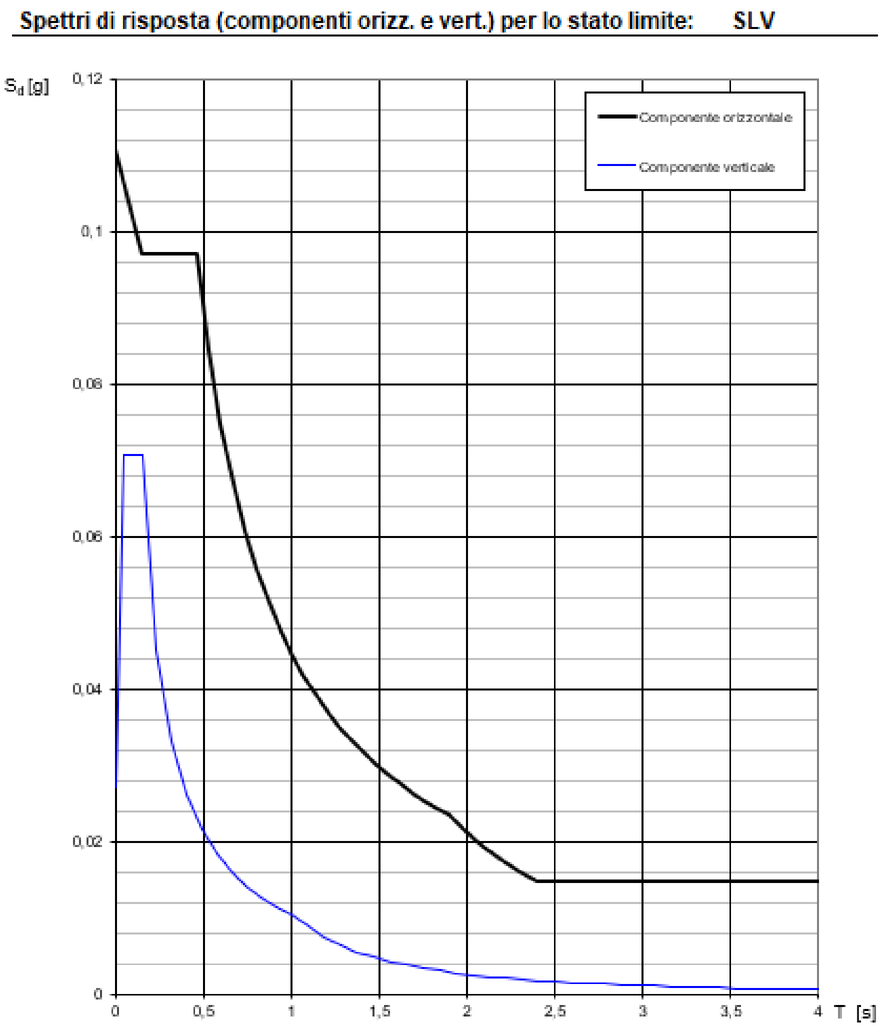

FASE 3. DETERMINAZIONE DELL'AZIONE DI PROGETTO
Stato Limite
Stato Limite considerato
SLV
info
Risposta sismica locale
Categoria di sottosuolo
C
info
S_S =
1,500
C_C =
1,576
info
Categoria topografica
T1
info
h/H=
0,000
S_T =
1,000
info
(h=quota sito, H=altezza rilievo topografico)
Compon. orizzontale
Spettro di progetto elastico (SLE)
Smorzamento ξ (%)
5
η =
1,000
info
Spettro di progetto inelastico (SLU)
Fattore q_o
2,975
Regol. in altezza
sì
info
Compon. verticale
Spettro di progetto
Fattore q
1
η =
1,000
info
Elaborazioni
Grafici spettri di risposta
Parametri e punti spettri di risposta
Spettri di risposta
$S_{d,o}$ [g]
$S_{d,v}$ [g]
S_e [g]
0,25
0,20
0,15
0,10
0,05
0,00
0
1
2
3
T [s]
4
Spettro di progetto - componente orizzontale
Spettro di progetto - componente verticale
Spettro elastico di riferimento (Cat. A-T1, ξ = 5%)
INTRO
FASE 1
FASE 2
FASE 3

For each of the phases described, it is also possible to view the graphs of the response spectra and the table of the parameters of the acting actions.

About "Phase 2", the table of parameter values associated with each limit state considered and the elastic response spectrum always relative to it are shown.

S. Limit	**T_R**	**a_g**	**F_0**	**T_{c^*}**
SLO	45,161	0,029	2,542	0,204
SLD	75,434	0,035	2,568	0,223
SLV	711,842	0,074	2,607	0,292
SLC	1462,179	0,092	2,617	0,301

In "Phase 3", on the other hand, it is possible to display the response spectrum, where both horizontal and vertical components are considered, only for the SLV limit state, which is the one that will then be considered in the seismic analysis.



Also related to 'Step 3' is the numerical representation of the parameters required for the analysis, which are divided into dependent and independent parameters.

Parametri indipendenti

STATO LIMITE	SLV
a_g	0,074 g
F_o	2,607
T_C^*	0,292 s
S_S	1,500
C_C	1,576
S_T	1,000
q	2,380

Parametri dipendenti

S	1,500
η	0,420
T_B	0,153 s
T_C	0,460 s
T_D	1,896 s

Espressioni dei parametri dipendenti

$S = S_S \cdot S_T$ (NTC-08 Eq. 3.2.5)

$\eta = \sqrt{10/(5+\xi)} \geq 0,55;\ \eta = 1/q$ (NTC-08 Eq. 3.2.6; §. 3.2.3.5)

$T_B = T_C / 3$ (NTC-07 Eq. 3.2.8)

$T_C = C_C \cdot T_C^*$ (NTC-07 Eq. 3.2.7)

$T_D = 4,0 \cdot a_g / g + 1,6$ (NTC-07 Eq. 3.2.9)

	T [s]	Se [g]
	0,000	0,111
T_B	0,153	0,097
T_C	0,460	0,097
	0,529	0,085
	0,597	0,075
	0,665	0,067
	0,734	0,061
	0,802	0,056
	0,871	0,051
	0,939	0,048
	1,007	0,044
	1,076	0,042
	1,144	0,039
	1,212	0,037
	1,281	0,035
	1,349	0,033
	1,417	0,032
	1,486	0,030
	1,554	0,029
	1,622	0,028
	1,691	0,026
	1,759	0,025
	1,827	0,024
T_D	1,896	0,024
	1,996	0,021
	2,096	0,019
	2,196	0,018
	2,297	0,016
	2,397	0,015
	2,497	0,015
	2,597	0,015
	2,697	0,015
	2,798	0,015
	2,898	0,015
	2,998	0,015
	3,098	0,015
	3,198	0,015
	3,299	0,015
	3,399	0,015
	3,499	0,015
	3,599	0,015
	3,699	0,015
	3,800	0,015
	3,900	0,015
	4,000	0,015

5 GEOMETRIC MODELING

Before proceeding with the creation of the model for finite element analysis, it is necessary to develop an architectural model as accurate as possible, to try to have the results of the analysis as reliable as possible.

For the creation of the architectural model, it was chosen to use Rhinoceros, a free-form 3D CAD modeling software. The function of Rhino is *to "create mathematical representations from the surfaces of an object. The resulting model appears on the screen as a two-dimensional image. Rhino provides the elements to create, show and manipulate these surfaces."*[7]

There is also the possibility to use different plug-ins to extend the possibilities of using Rhino and expand its functionality.

3D Model

To model the structure of the church under analysis, the previously available DWGs were initially modified, referring to the measurements made on site with the previously described tests. To simplify the modelling, the entire church is represented as a set of volumes, while the vaults were created as surfaces.

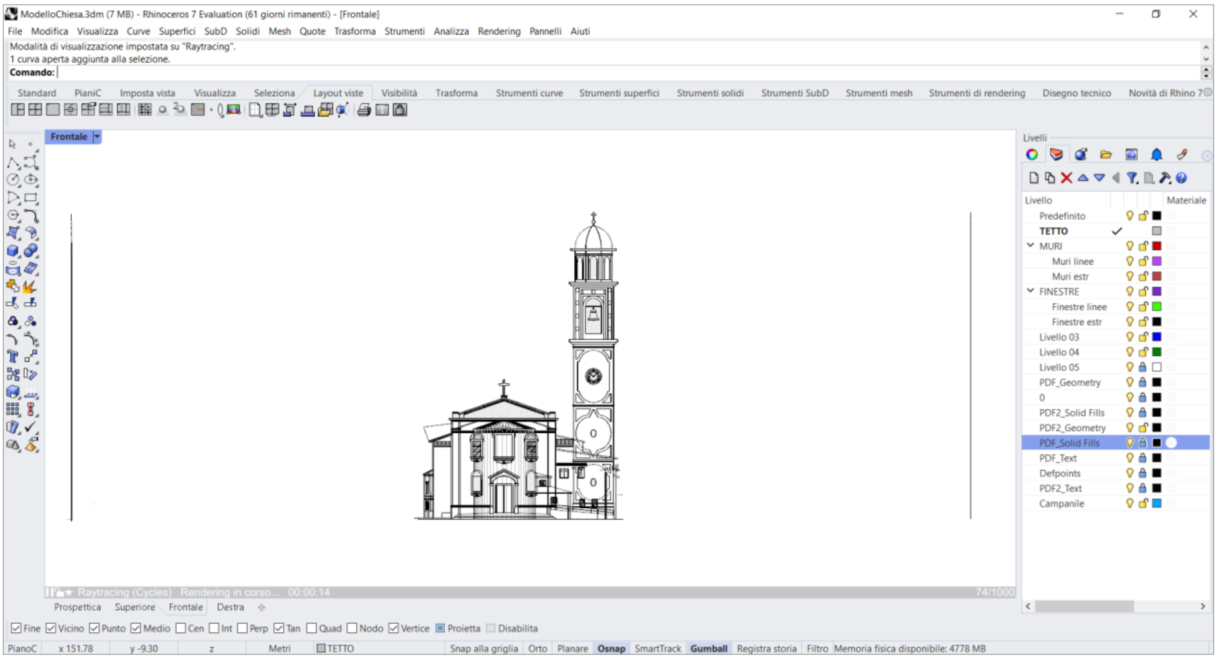
After editing the relevant measurements and masonry thicknesses, four different DWG files were imported into Rhinoceros to facilitate the creation of the three-dimensional model and to facilitate the creation of volumes and surfaces.

The four imported DWGs are:

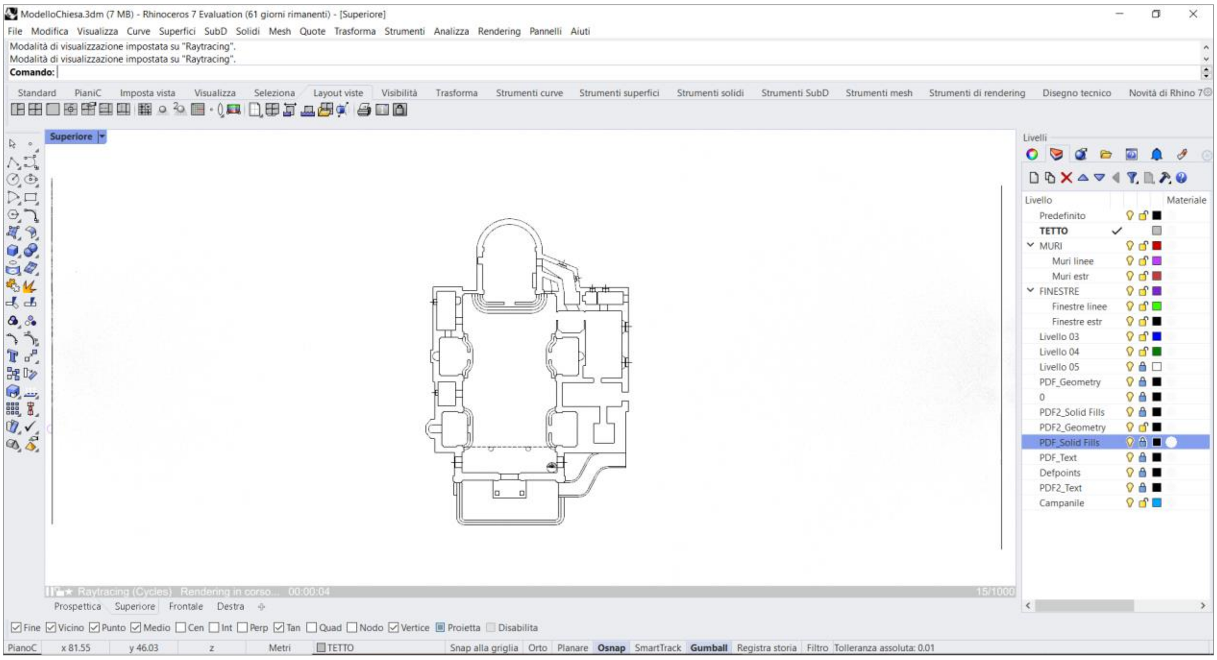
- The building plans.

- The four elevations.

[7] Rhinoceros Help - Rhinoceros User Guide



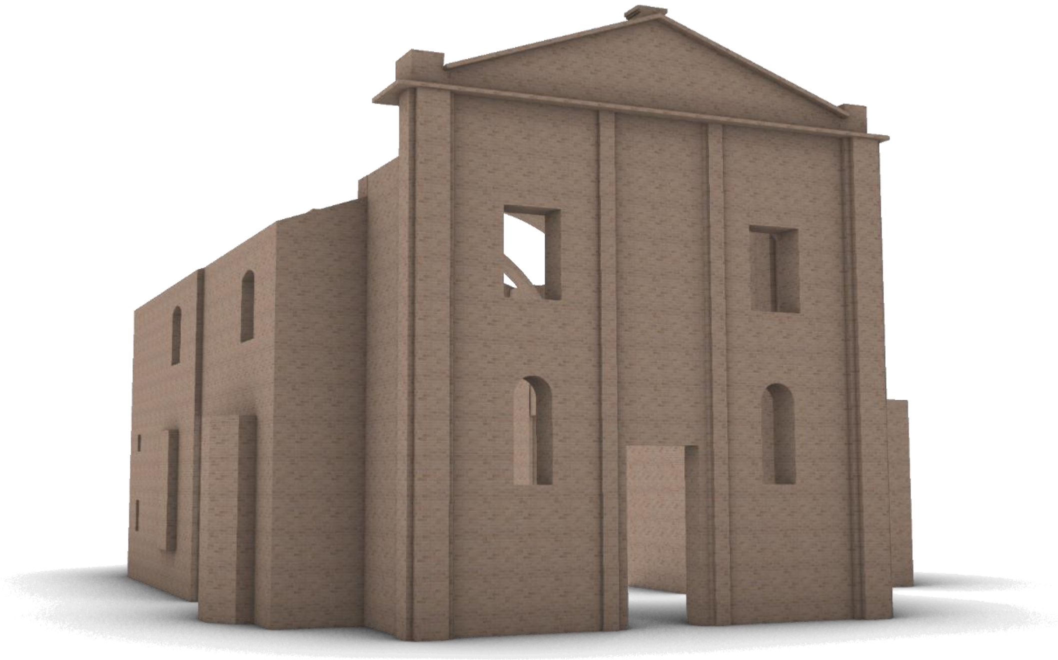
The inserted DWG files were called "PDF_SolidFills" as shown in the figure. All four elevations were inserted, but only the front elevation is shown. The ground plan was placed in the centre of the elevations and aligned with them.

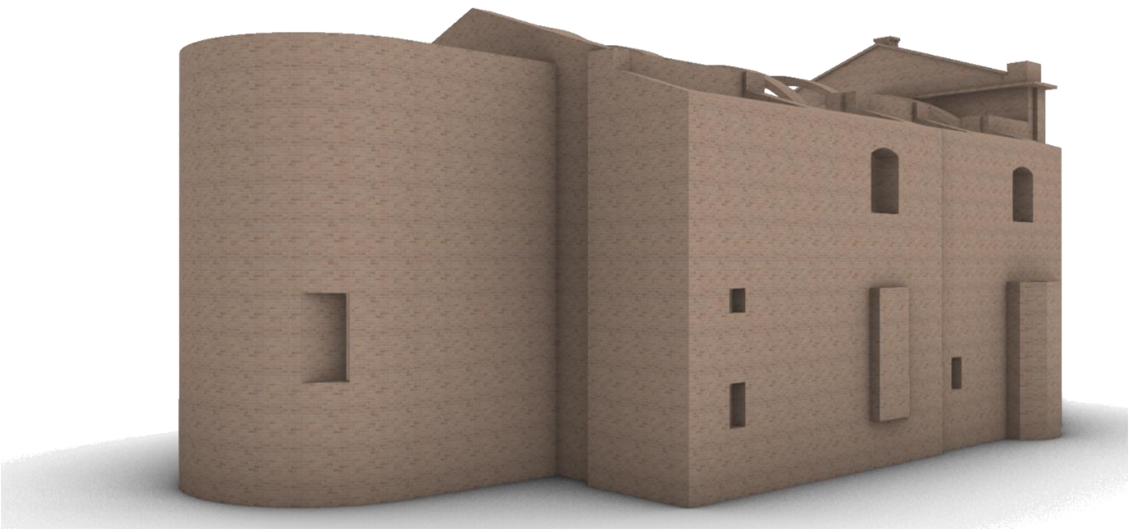


The final illustration shows how the four files were introduced into Rhinoceros to facilitate the creation of the three-dimensional model.

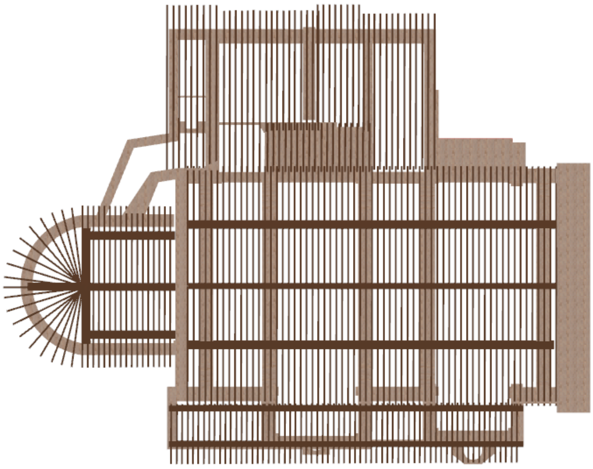
5.1 Masonry

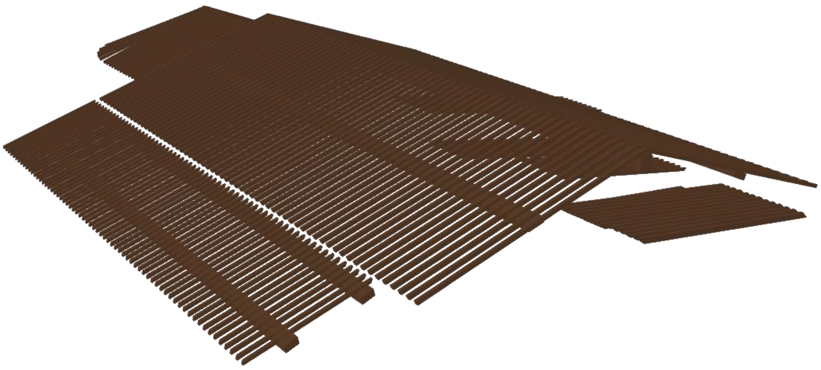
First, after defining all the thicknesses of the different walls, the perimeter walls were raised which have different heights on the façade and on the sides, as the façade has a greater height, as can be seen from the three-dimensional view below.

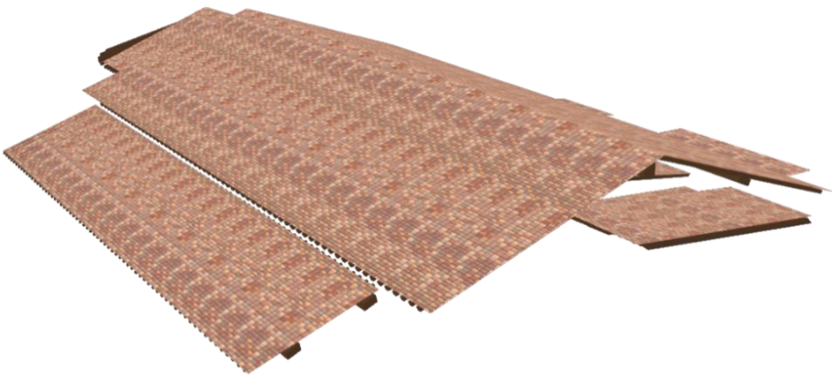




5.2 The roof







5.3 The bell tower

The bell tower is considered independently because, as told in the history of the church, initially it was placed in a different location; years after construction, it was decided to build the new bell tower that coincides with the current one, as reported in the figure below.

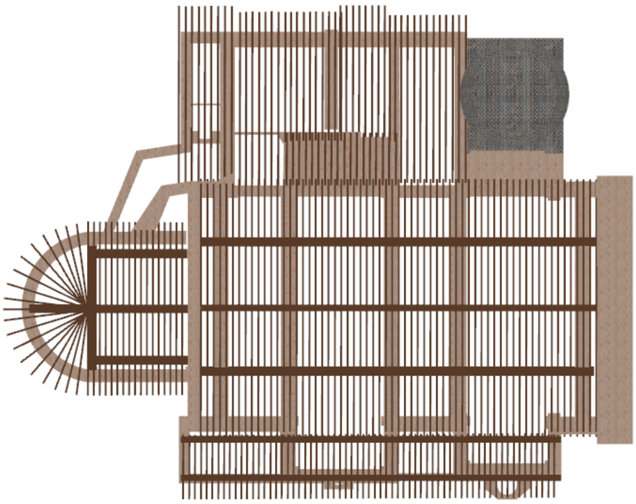
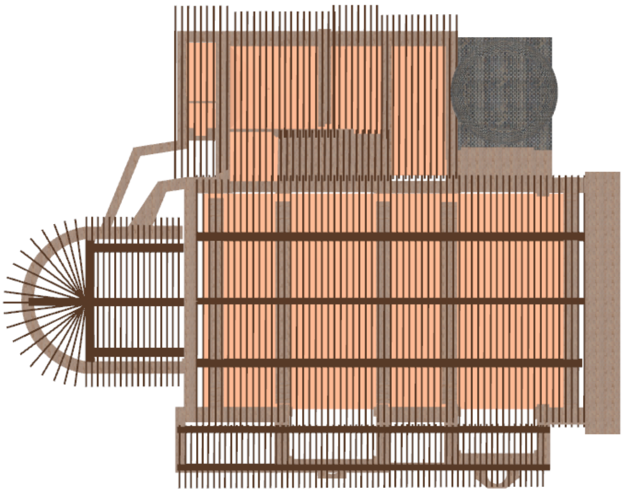


Figure 43 - Bell Tower, Top View

Figure 44 - Bell Tower Complete Model

5.4 The vaults

To model the vaults in the church, it was decided to model them as individual elements in AutoCad they were then inserted into Rhino and modelled as surfaces.



The first vault that has been considered and studied is the vault of the sacristy. For other internal covers, such complex covers have not been considered at times but have been simplified to try to facilitate the final analysis of the model. Roofs with simple barrel vaults were thus considered (indicated in orange in the plan).

5.5 Render of the complete architectonic model

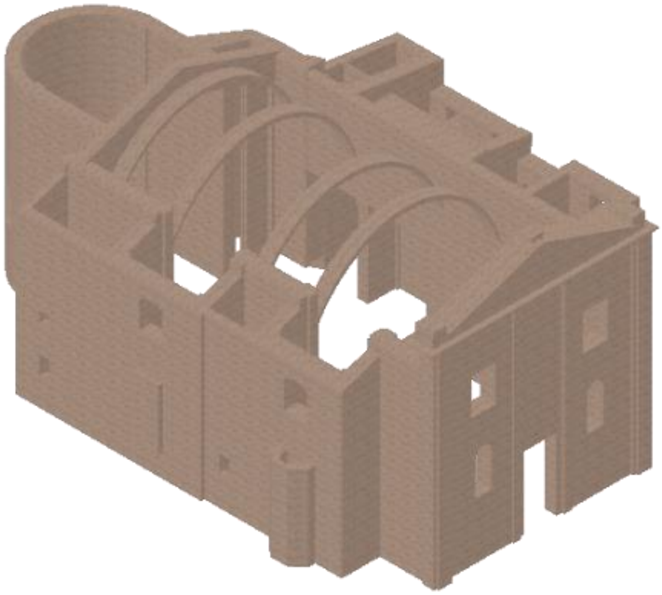


Figure 45 - Architectonic Model, Structural Wall and Facade

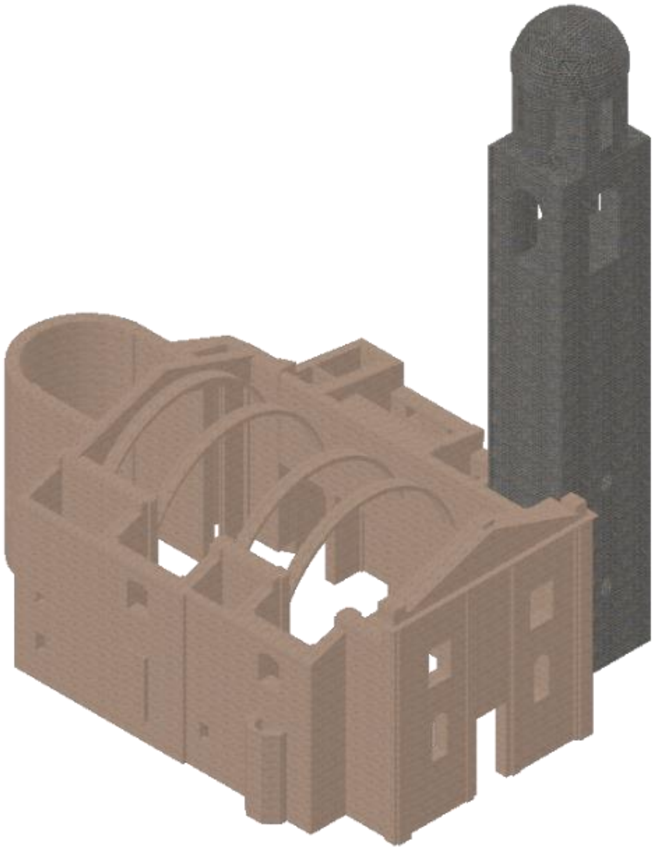


Figure 46 - Architectonic Model, Structura Wall, Facade and Bell Tower

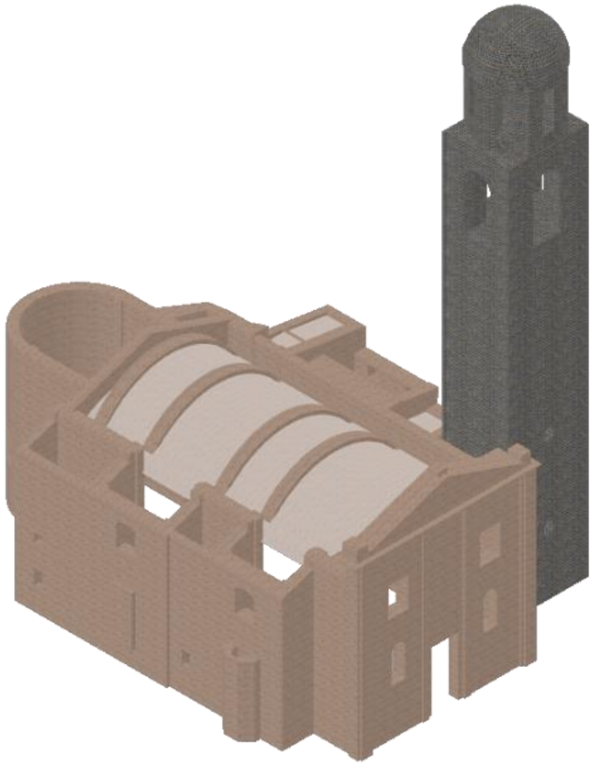


Figure 47 - Architectonic Model with the vaults

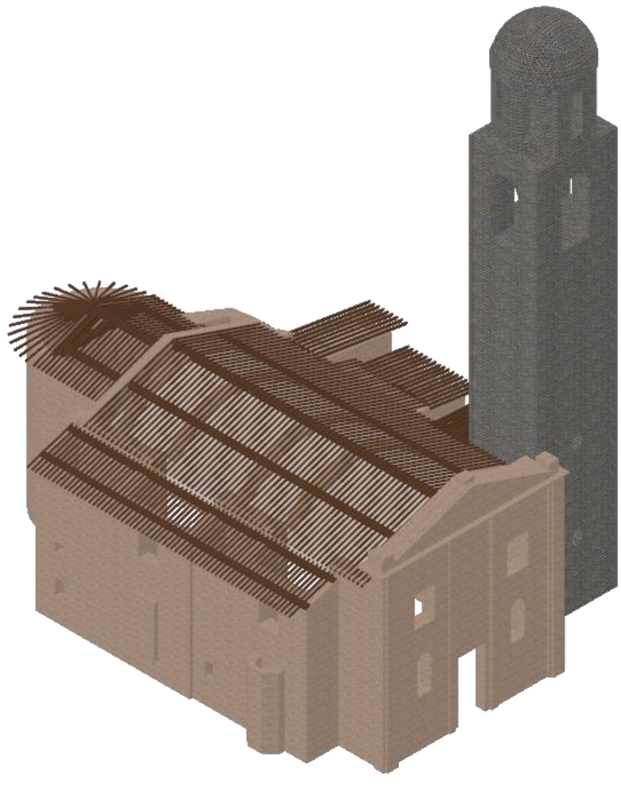


Figure 48 - Architectonic Model with the Structure of the Wood Roof

Figure 49 - Complete Model, Frontal View

Figure 50 - Final Architectonic Model, Rhino 3D

6 FEM MODELING

For the creation of the structural model, to be used for the subsequent analysis, the Abaqus/CAE software was used; “Abaqus" is a software suite for finite element analysis and computer-aided engineering, originally released in 1978; with this software it is possible to model, analyse and visualize a structure with the finite element method and display results in also in graphic form.

The following are the main steps taken to create the model. Compared to the previously presented architectural model, some simplifications were made to reduce the complexity of the geometry. The cover was not included in the analysis, only the own weight as load.

The blocks exported from Rhino and imported as solids into Abaqus were as follows:

- Central body, supporting structure.
- Arches.
- Façade.
- Bell tower.

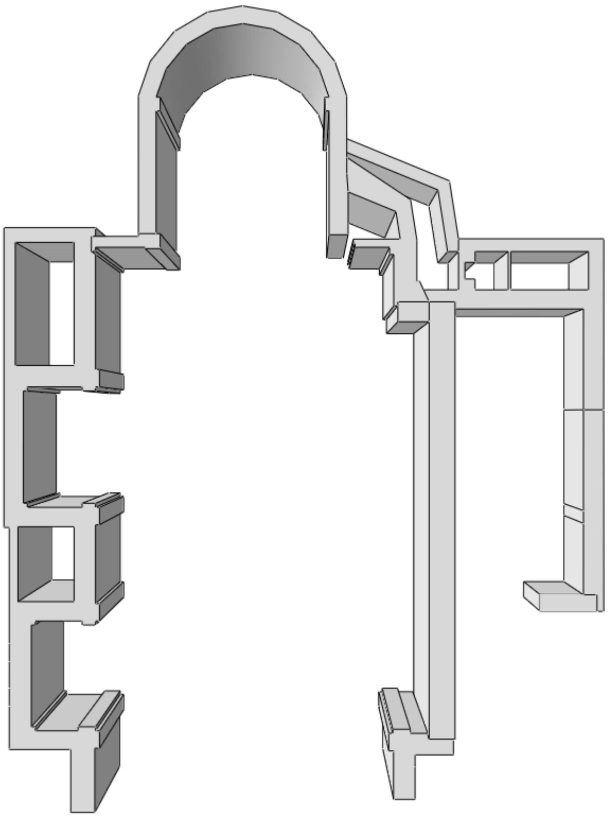


Figure 51 - Block 1 imported in Abaqus

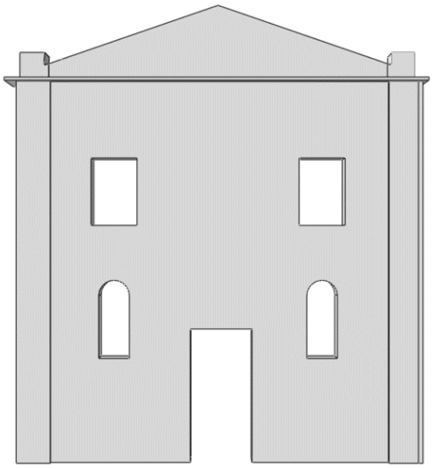


Figure 52 - Block 2 imported in Abaqus

After all blocks were imported as 'Part', the specific material for each block had to be defined. For the supporting structure, the arches and the façade, the material is constant; in addition, a 'Section - Homogeneous and Isotropic' is assigned as the analysis will initially be linear elastic. Both for the structural wall and for the façade, the type of masonry in solid bricks and lime mortar was chosen.

Once the model is assembled, the 'BC - Boundary Conditions' are set, shown in the figure below, and the gravity is applied to the whole structure.

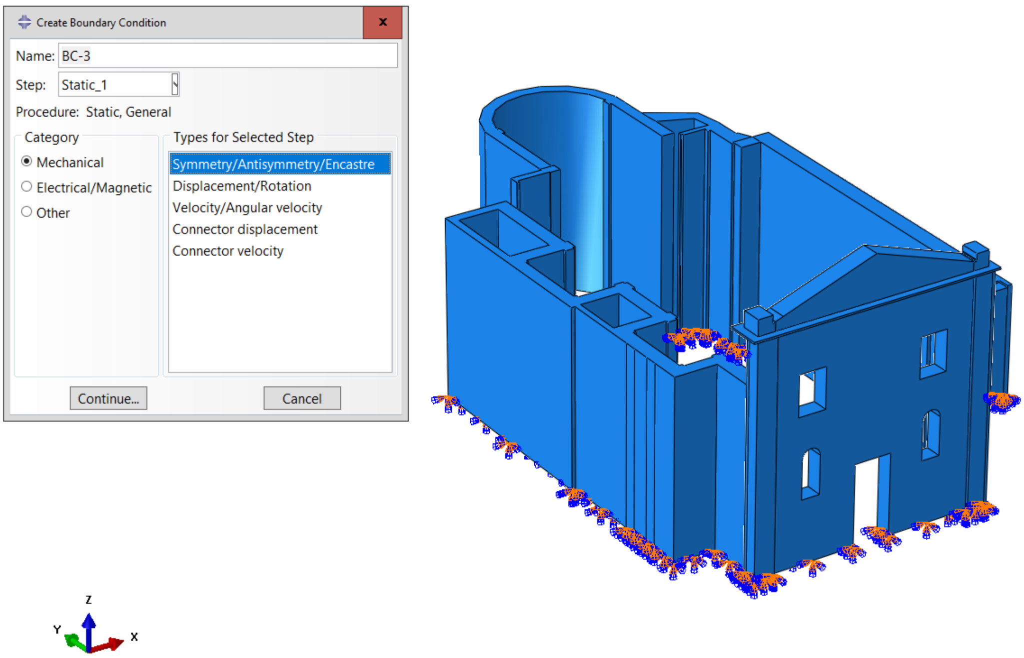


Figure 53 - Boundary Condition - Assembly Block 1 and Block 2

After the definition of the materials and of the sections, the “Assembly” step must be done, in order to create an only part, composed by the façade and the walls.

For the static analysis, the following load has been applied in order to evaluate the self-weight of each singol elements.

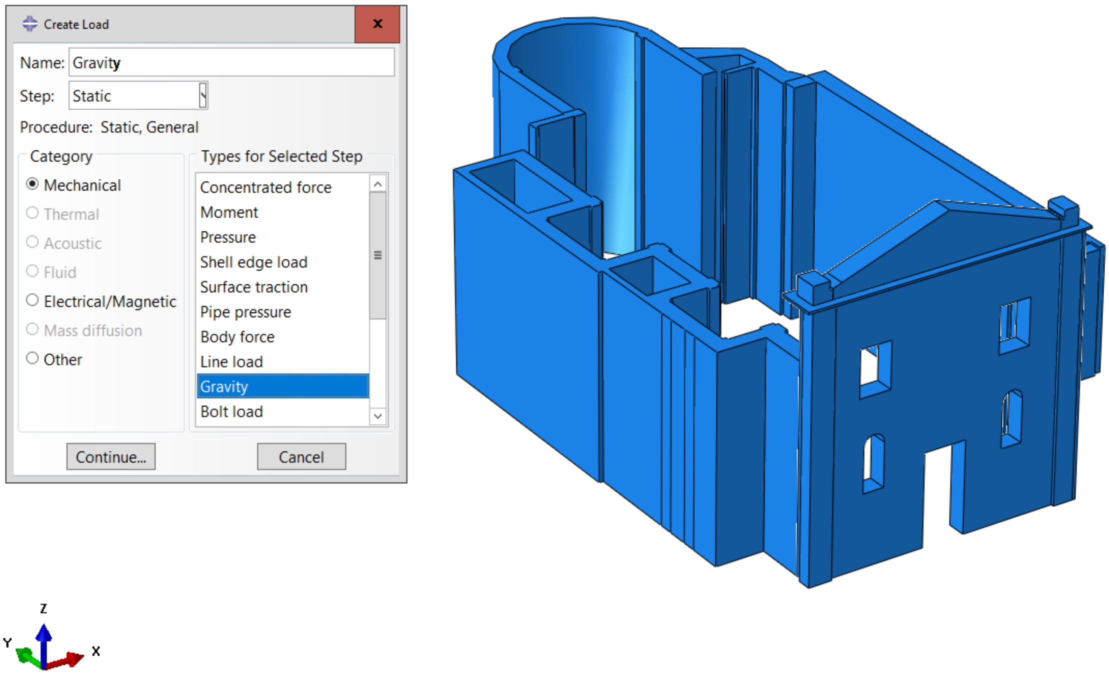


Figure 54 – Gravity Loads applied to the model

After the application of the main load, the creation of the mesh must be done; this part is very important for the simulation, in fact it is necessary to choose the correct typology of mesh, depending on the considered element, to have good and precise results.

For this reason, for all the parts of the church, except for the vaults, a Tet-Lin mesh, with 0.8 measure of the seeds, is considered, this is because this typology can adapt to different typology of shapes and structures. Initially only the two "Blocks" were meshed as single elements, and finally the entire part has been meshed as follow:

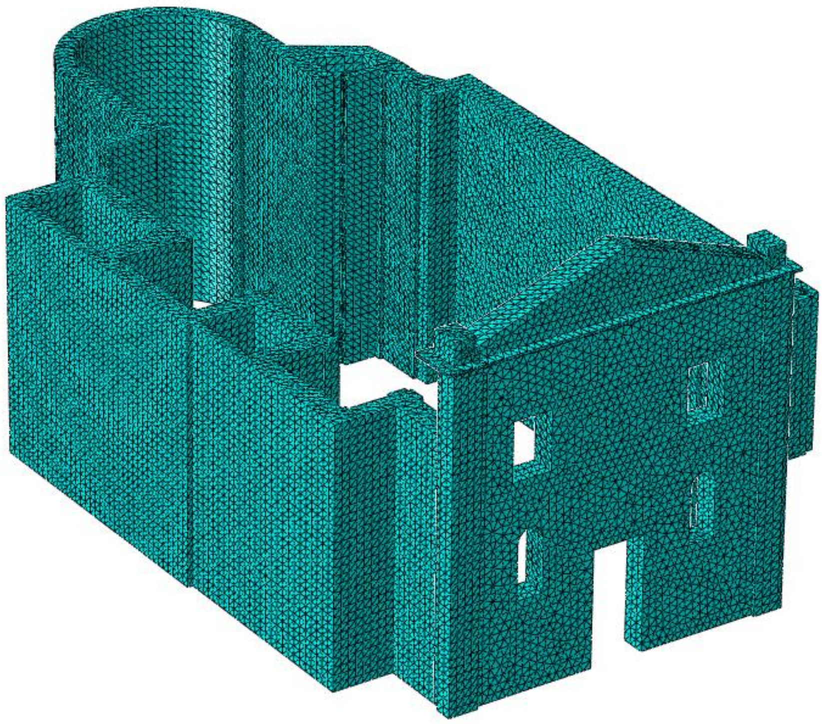


Figure 55 – Mesh Creation

The same steps are done to add to this model the other parts, the bell tower and the vaults; after a first analysis, the "Block 1" It was replaced considering the main walls together with the arches, as the only part.

7 ANALYSIS

7.1 Static Analysis

7.1.1 Structural Wall and Façade

The first check was a static analysis only on the load-bearing structure connected to the church façade, without importing the vaults of the arches and the bell tower.

The static analysis is carried out by applying only the structure's self-weight. Below are the two main results to be analysed for this first configuration.

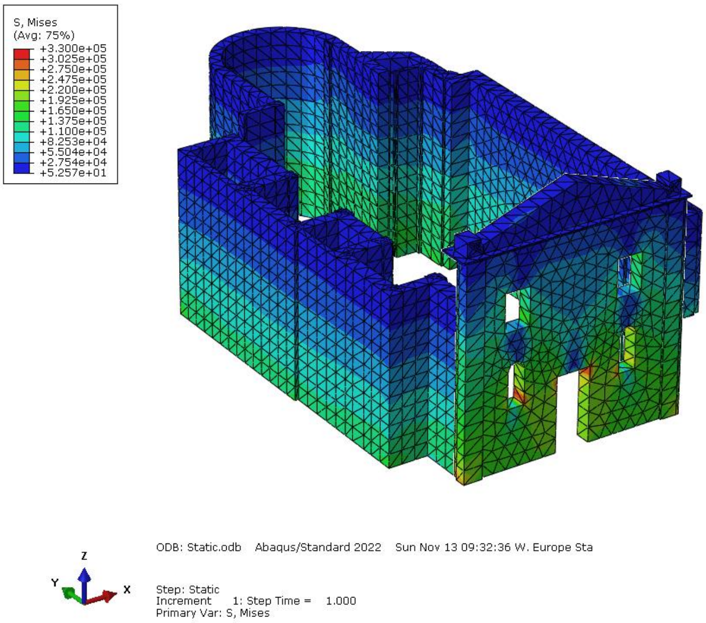


Figure 56 – Static Analysis, S Mises

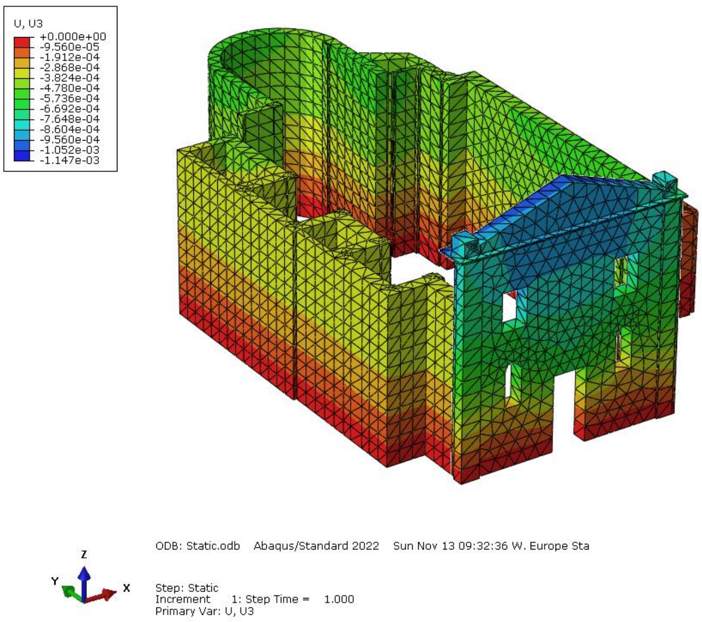


Figure 57 – Static Analysis, U3

7.1.2 Bell Tower

Since the steeple will also be subjected to linear analyses such as modal analysis and response spectrum, a static analysis was also carried out for this model. It is thus possible to study the internal stresses and deformations to which the steeple is subjected.

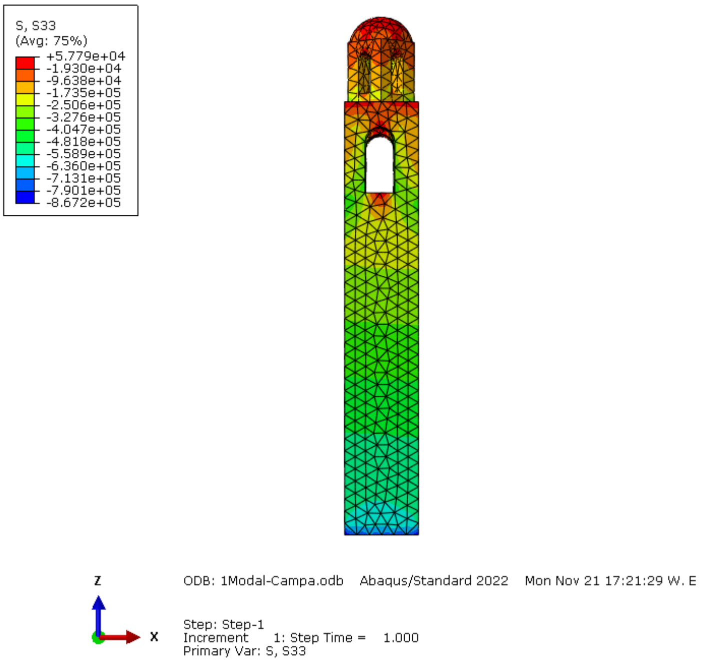


Figure 58 – S. Analysis, Bell Tower, S33

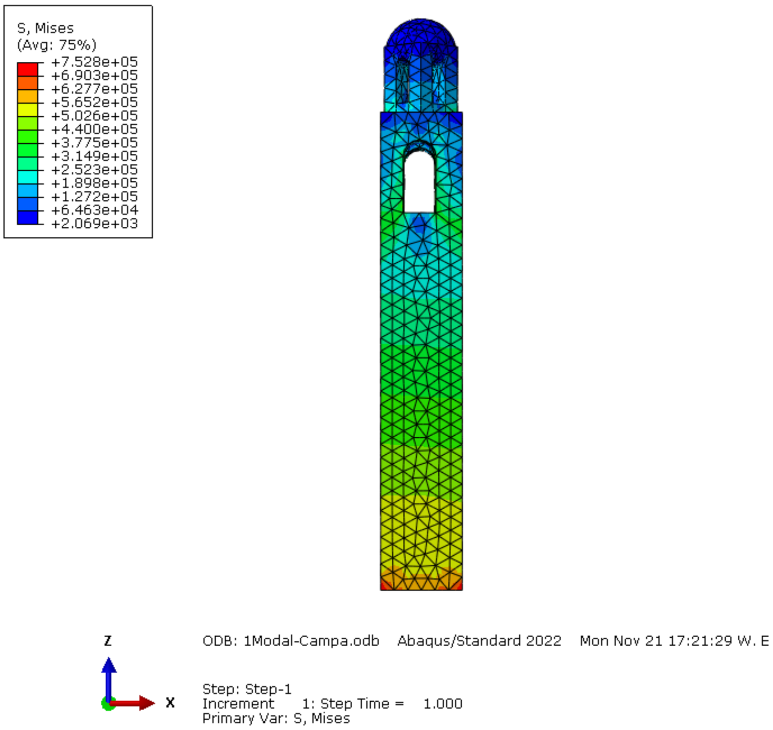


Figure 59 – S. Analysis, Bell Tower, Mises

7.1 Modal Analysis

Modal analysis is carried out to study the behaviour of the structure considered when subjected to vibration, thus being able to determine the natural frequencies of oscillation when the structure is subjected to free oscillation.

Three different modal analyses were carried out for the structure, initially only the load-bearing walls, the arches and the façade were considered, then the bell tower was added, and in the last step the vaults were included, relating only to the nave.

As for the roof, it was considered as a weight and was not modelled.

7.1.1 Church (Structural Wall, Arches and Façade)

Initially, to verify that the behavior of the structure was appropriate to the type, a modal analysis was performed only on the union of the "Block Walls & Arches" and the "Block Facade", without considering the other parts.

Only the first four modes will be reported, to understand if the results, in terms of period, are significant and sensible. The other modes will be reported in Appendix A.

Mode 1
T [s]: 0.26

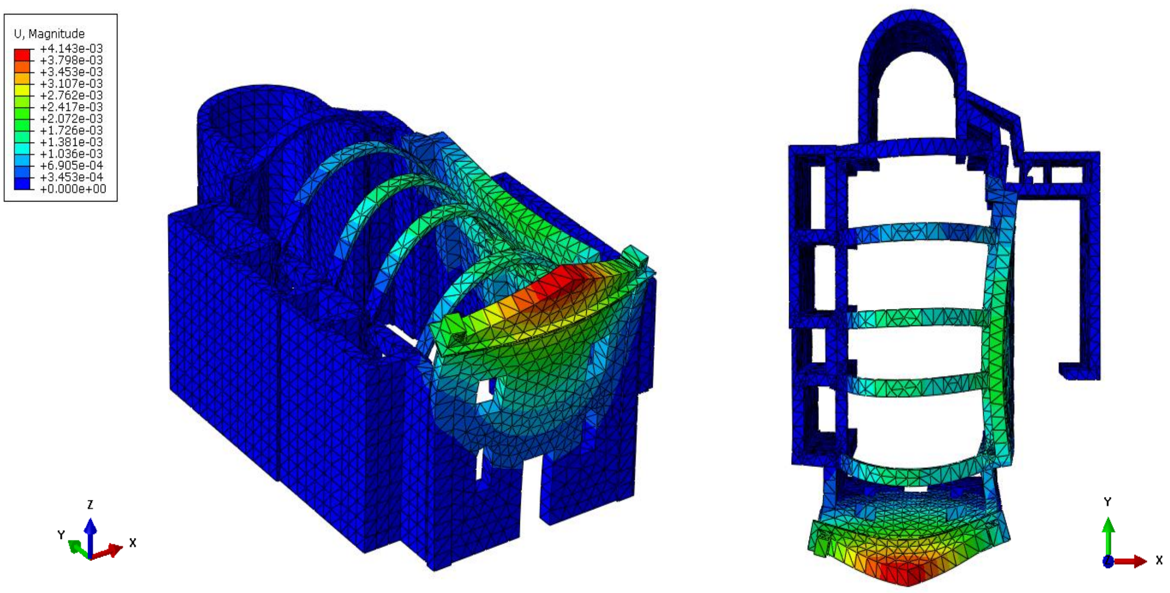


Figure 60 – Modal Analysis, Mode 1

From this first mode we can see that the top façade is not well constrained by the rest of the structure, as it is the first one to show a large deformability that may bring to damage mechanism; the arches in this case without the vaults are free to move and the right wing of the nave being constrained only to the arches shows a behaviour that is usually seen from the 6th or 7th mode.

Mode 2

T [s]: 0.22

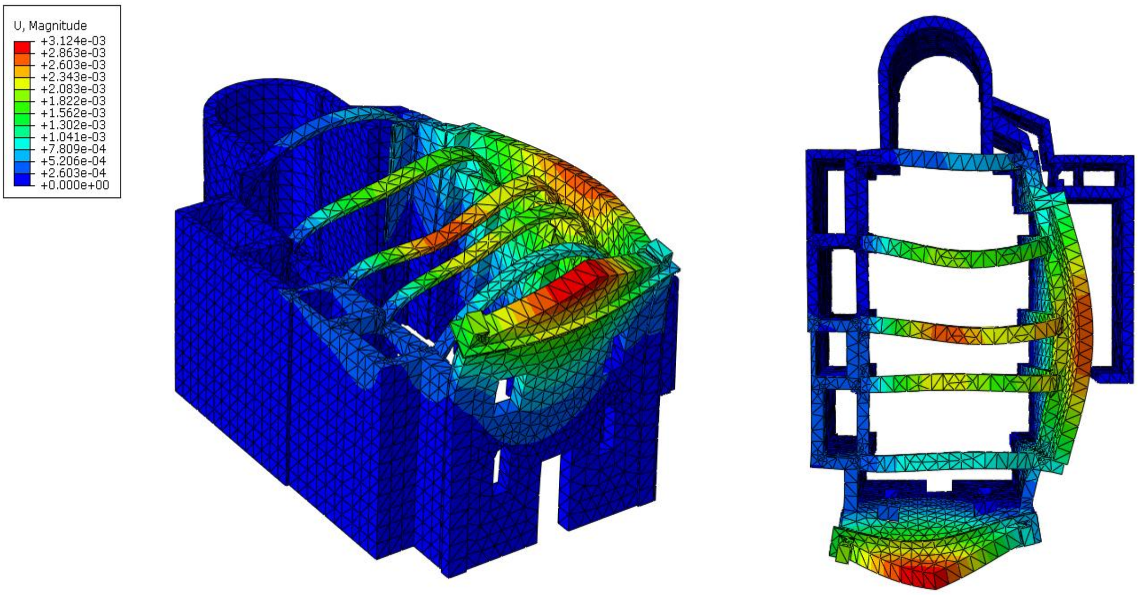


Figure 61 – Modal Analysis, Mode 2

Again, it is remarkable how the façade and the response of the right-side wing of the nave are very high, obviously this is due to the absence of the vaults and the bell tower, which, as we will see in the following analysis, contribute greatly to the stability of the structure. Obviously, the period was reduced, although not by much. It can be seen, of course, that the right side of the church is more prone to deformation and for this reason the presence of a stiffening structure is necessary. The bell tower was added later; as we will see in the following analysis, it provides more stability to the right wing, but of course this only applies if the connection between the two is effective.

Mode 3

T [s]: 0.18

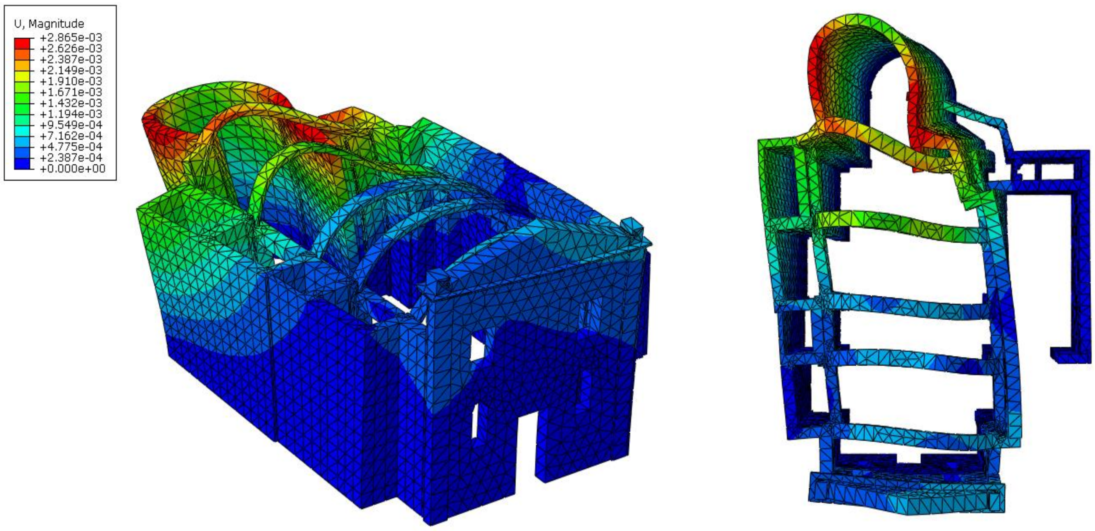


Figure 62 – Modal Analysis, Mode 3

In this third mode, the mechanism no longer involves the façade, but rather the arches at the end of the nave and the apse; we will see in later analyses how the insertion of the vaults will change this behaviour.

Mode 4

T [s]: 0.16

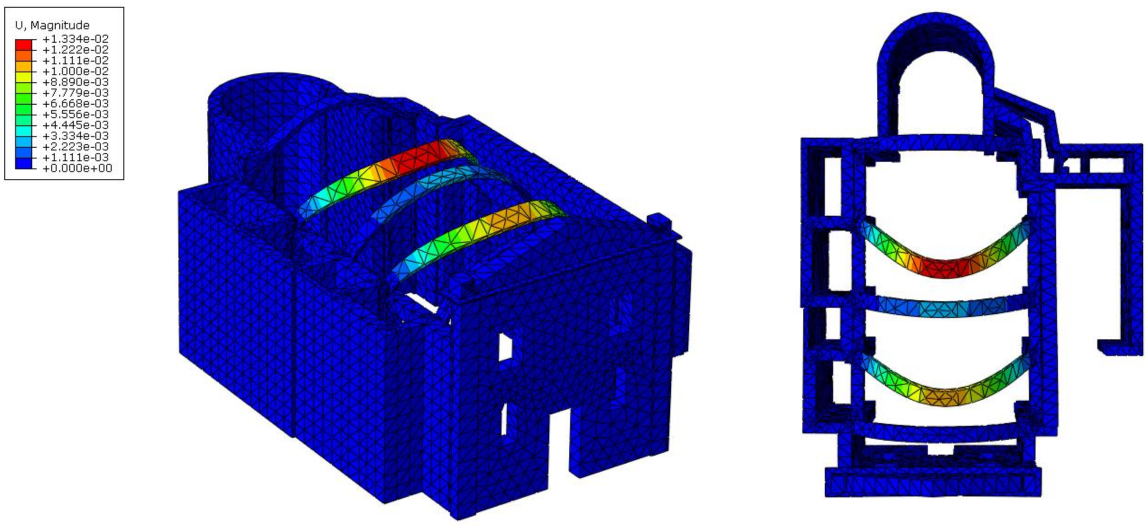


Figure 63 – Modal Analysis, Mode 4

This is a very unusual mode, which only concerns a displacement of the arcs; it will no longer appear after the insertion of the vaults which will stiffen the central part.

7.1.2 Bell Tower

Before proceeding with the analysis of the complete model, the results of the modal analysis carried out on the individual bell tower are presented.

This choice is due to the fact that, as described in the historical chapter, the bell tower was added at a later date than the construction of the entire building and, for this reason, even with respect to the tests carried out in situ (with results that were not entirely positive in terms of thermography and sonic tests) the bell tower does not appear to be well matched with the rest of the structure as the masonry was added at a later date, not integrating it with the part already existing. It was therefore decided to initially study only the belfry to see its response and the periods for this single structure and then to integrate it with the rest of the structure to see the change this implies on the load-bearing structure and the façade in terms of modes and periods.

For the belfry, the first 20 modes were studied, which are reported below. From the Abaqus software all the values of the frequencies related to the modes were exported, from here the period was calculated by inverse formula and the values of the participating masses in the different directions were exported. Only the main modes are shown graphically; the others will be listed in Appendix B.

	Frequency	**Period [s]**
1	0,788	1,269
2	0,790	1,266
3	3,551	0,282
4	3,562	0,281
5	4,243	0,236
6	7,403	0,135
7	8,311	0,120
8	8,348	0,120
9	8,461	0,118
10	10,809	0,093
11	11,023	0,091
12	15,182	0,066
13	17,388	0,058
14	17,787	0,056
15	17,943	0,056
16	18,427	0,054
17	21,761	0,046
18	26,155	0,038
19	26,445	0,038
20	26,735	0,037

Mode 1

F [Hz]: 0.7879

T [s]: 1.269

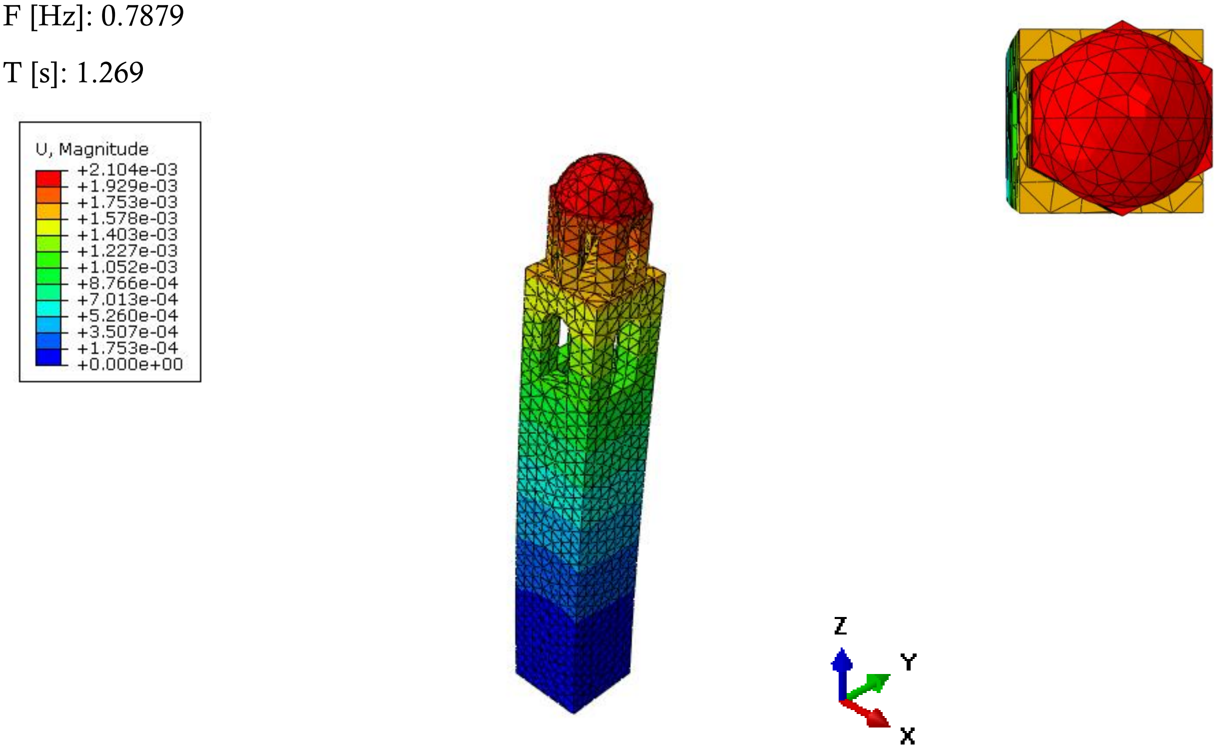


Figure 64 – Modal Analysis, Bell Tower, Mode 1

Mode 2

F [Hz]: 0.7902

T [s]: 1.2656

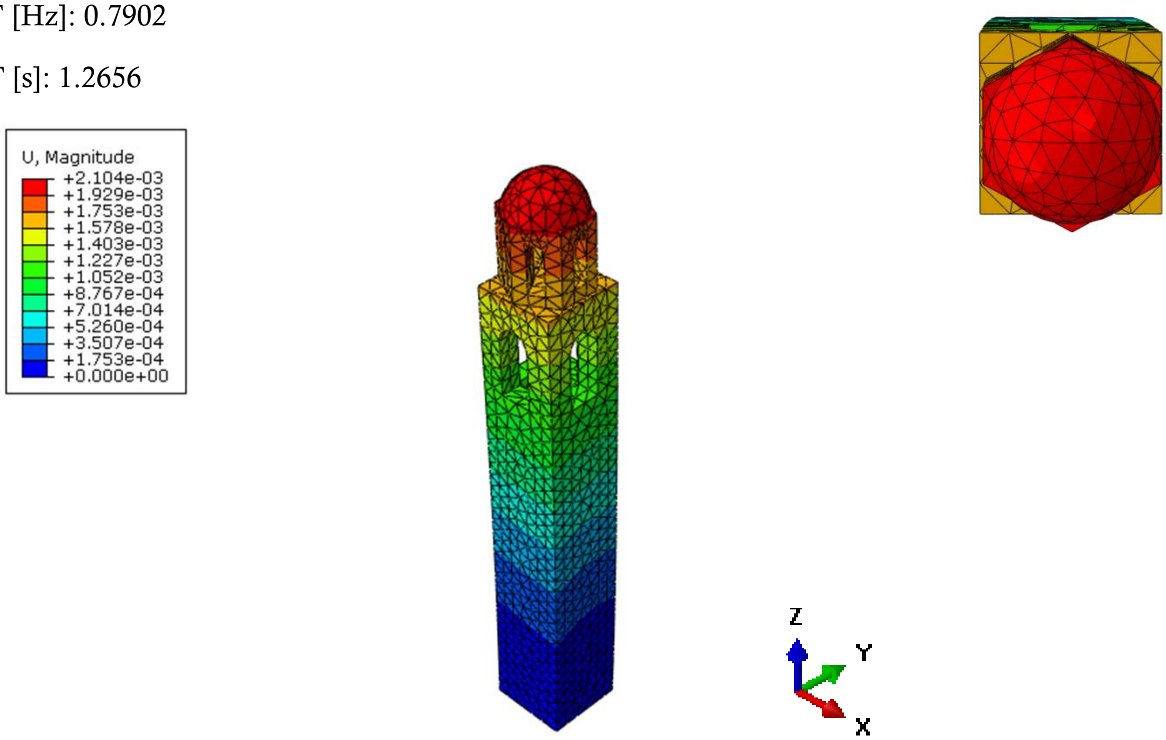
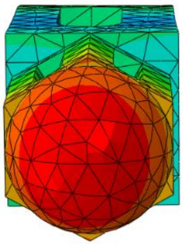


Figure 65 – Modal Analysis, Bell Tower, Mode 2

Mode 3

F [Hz]: 3.5505

T [s]: 0.2817



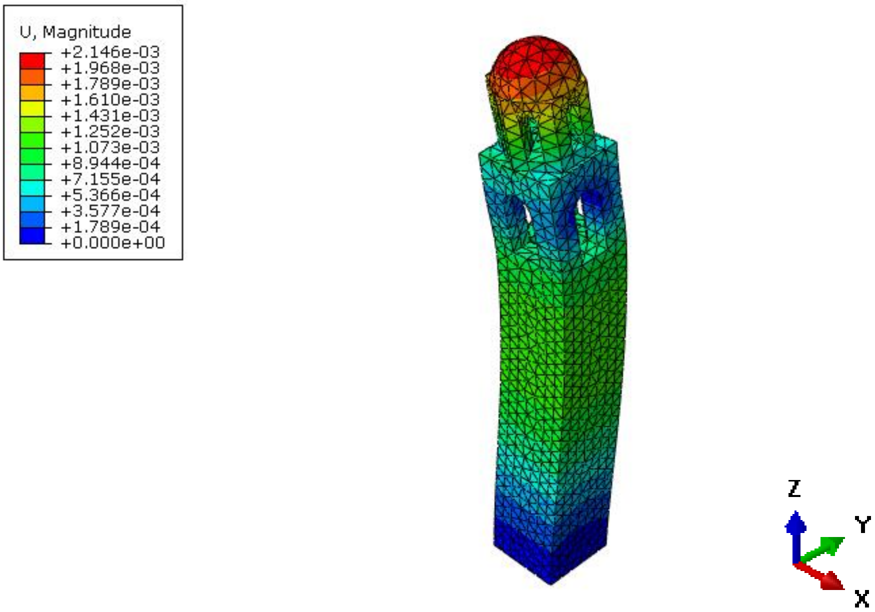
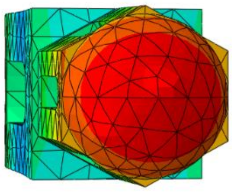


Figure 66 – Modal Analysis, Bell Tower, Mode 3

Mode 4

F [Hz]: 3.5617

T [s]: 0.2808



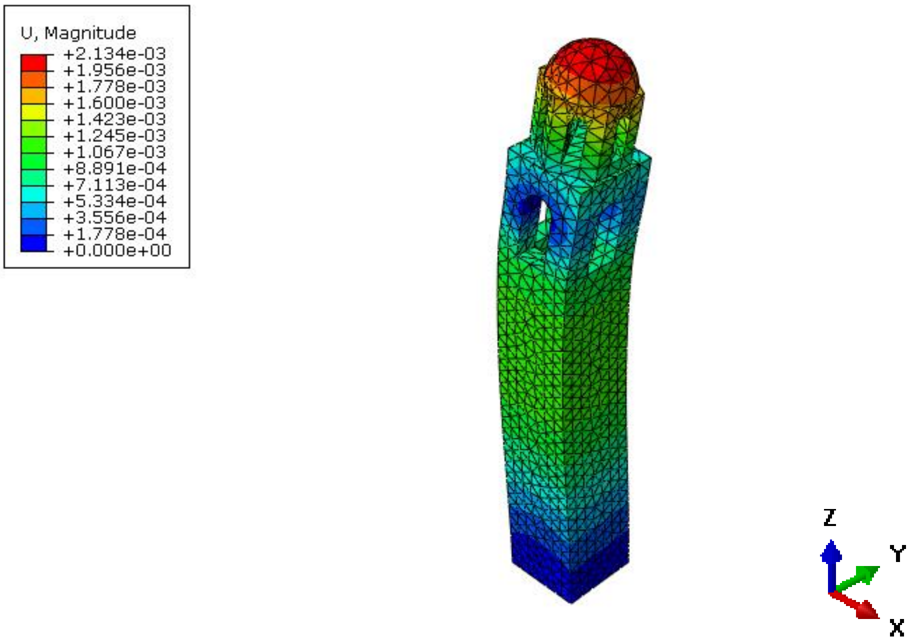


Figure 67 – Modal Analysis, Bell Tower, Mode 4

Mode 5

F [Hz]: 4.2432

T [s]: 0.2357

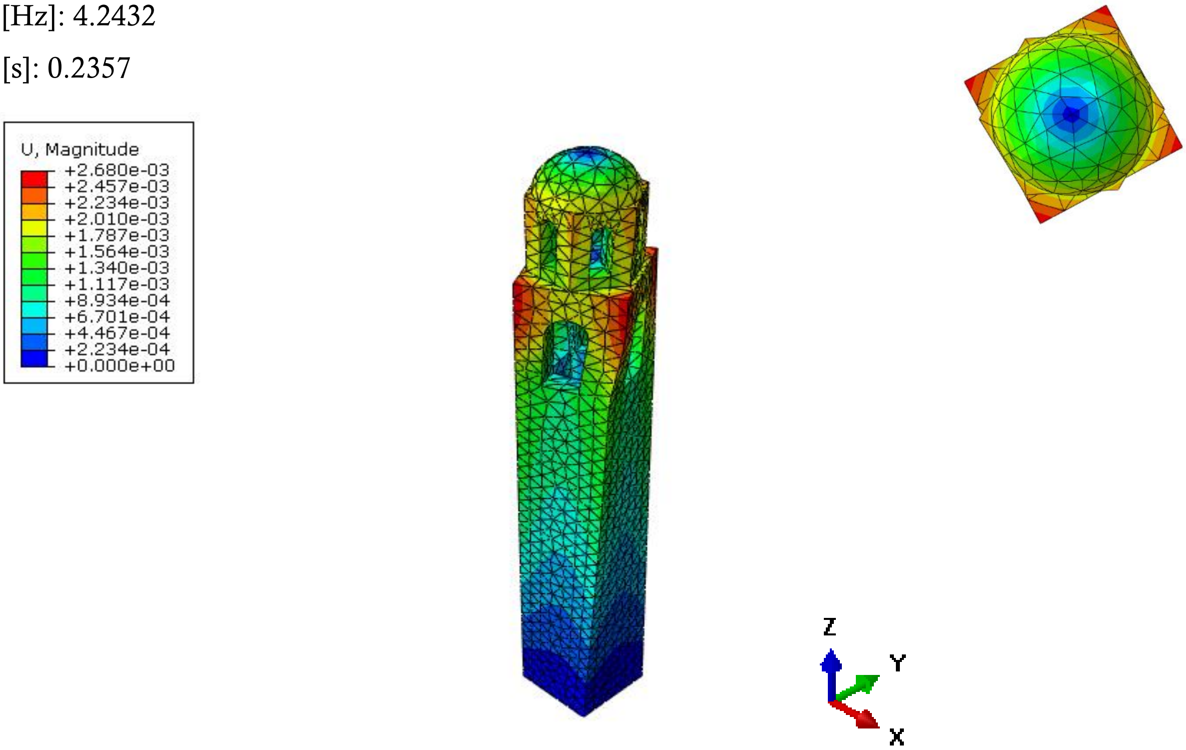


Figure 68 – Modal Analysis, Bell Tower, Mode 5

Mode 6

F [Hz]: 7.4027

T [s]: 0.1351

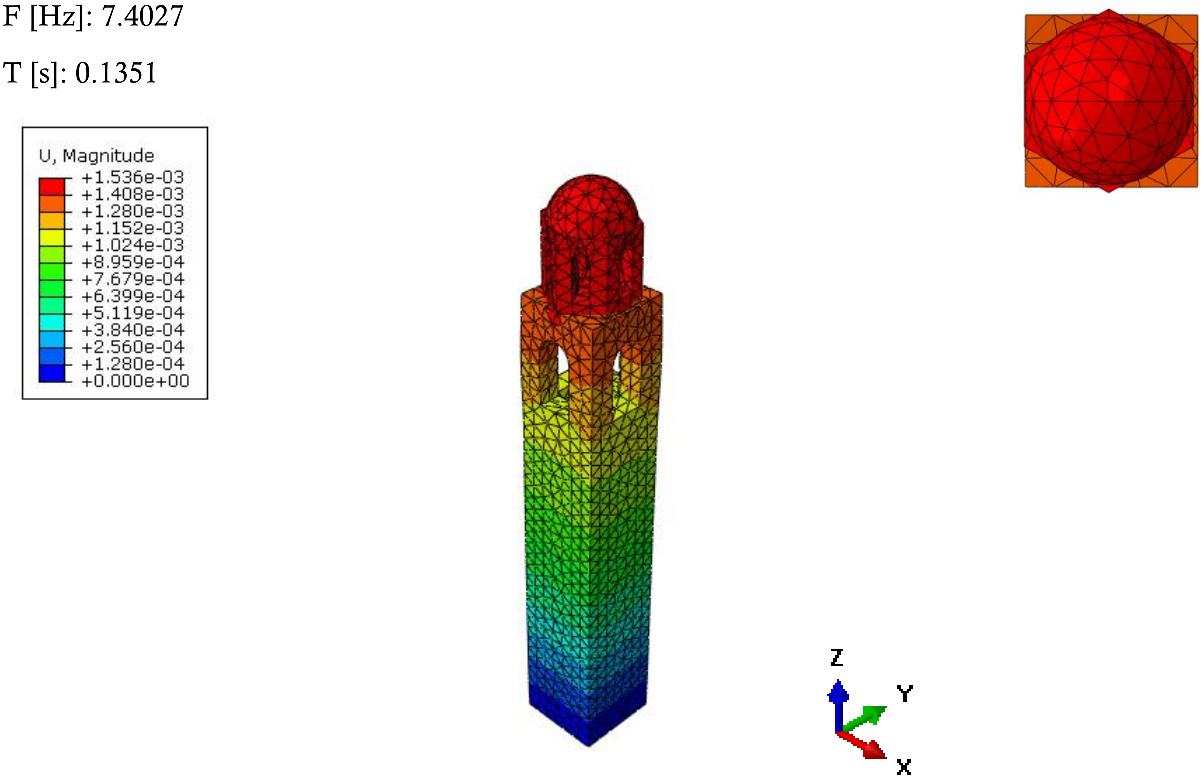


Figure 69 – Modal Analysis, Bell Tower, Mode 6

Mode 7

F [Hz]: 8.3113

T [s]: 0.12

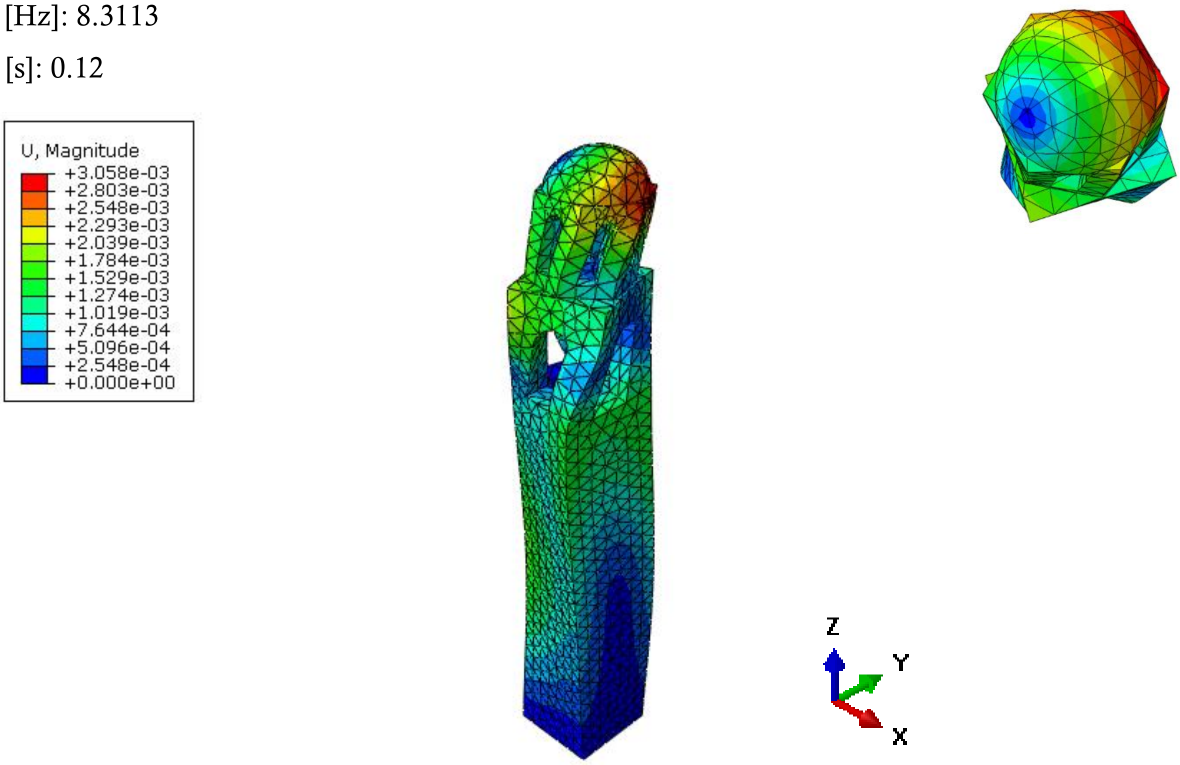


Figure 70 – Modal Analysis, Bell Tower, Mode 7

Mode 8

F [Hz]: 8.3482

T [s]: 0.119

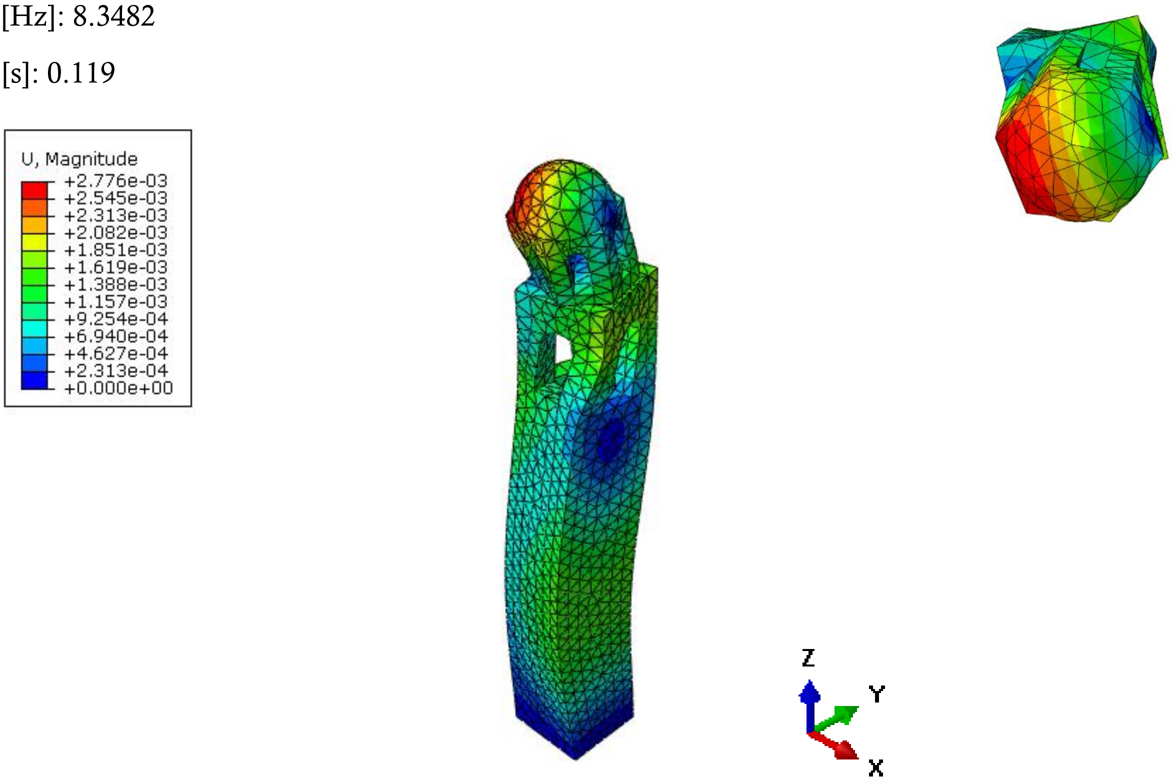


Figure 71 – Modal Analysis, Bell Tower, Mode 8

Mode 9

F [Hz]: 8.4612

T [s]: 0.118

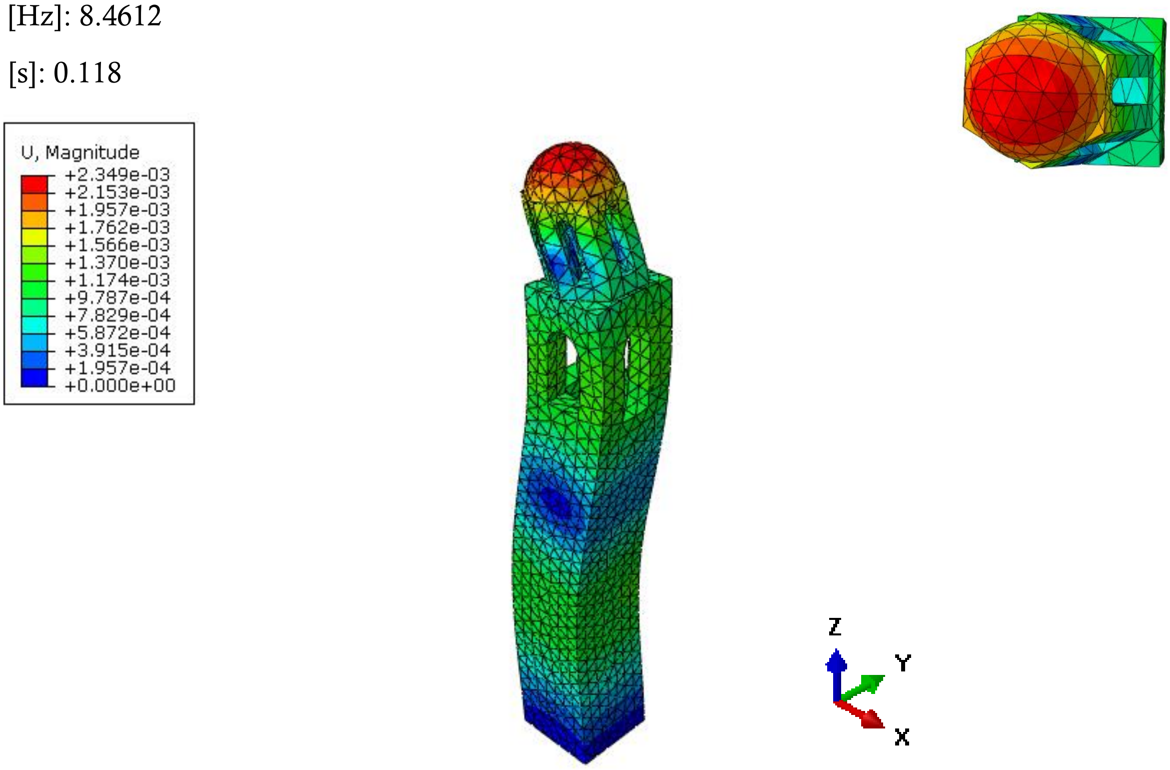


Figure 72 – Modal Analysis, Bell Tower, Mode 9

Mode 10

F [Hz]: 10.80

T [s]: 0.092

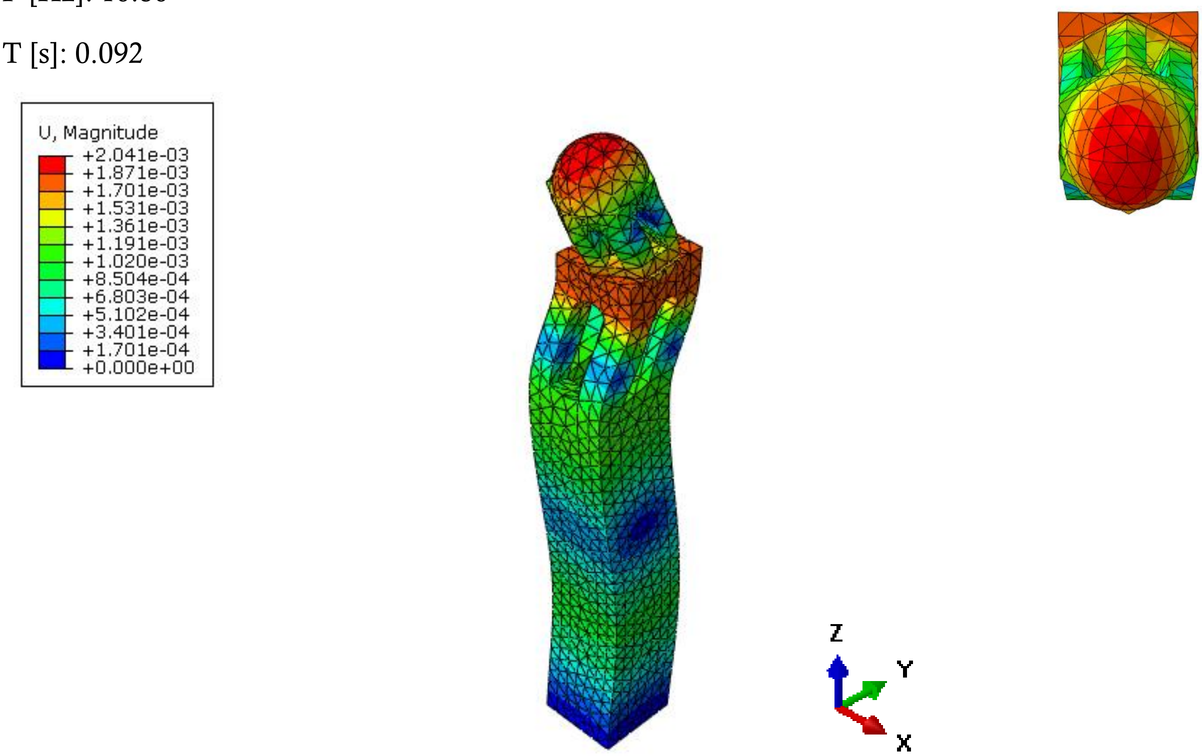
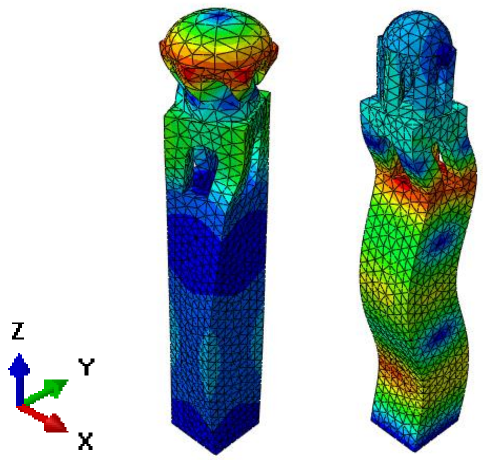


Figure 73 – Modal Analysis, Bell Tower, Mode 10

The first 10 modes of vibration of the bell tower have been shown above; as mentioned above, the remaining 10 are presented in Appendix B at the end of the thesis.

The bell tower has similar behaviour in the two directions, X and Y, since it has a symmetrical cross-section; in fact, the frequencies are very similar in each of the two modes. For the first and second modes, there is a displacement of the structure, with greater deformation at the top, in both directions; for these modes the period is very high, around 1.27 s, for the second direction it drops slightly but always remains around this value.

The third and fourth modes present a flexural form in X and Y, the fifth mode, on the other hand, presents the first torsion, around the top apertures, which turns out to be the predominant behaviour. It can be seen, however, that in all the modes shown, the part most subject to deformation is the top of the belfry, in correspondence with the four side openings and the upper lantern. Increasing the number of modes, in fact, the greatest deformations are at this point, thus affecting the top, the four perimeter columns of the lantern. For clarity, two modes are shown which represent what has been described.



To provide a comparison, we estimate the value of the fundamental vibration period. According to an empirical formula, often use, it is possible to evaluate the fundamental period of vibration based on an empirical equation, which depends on the type of structure that we are considering.

$$T_{LV1} = 0.013 \cdot H^{1.1} = 0.013 \cdot 37^{1.1} = 0.7\ s$$

$$T_{Abaqus\ (1°)} = 1.269\ s$$

The difference between the two differently calculated parameters is remarkable; the empirical formula is usually used to study very simple buildings.

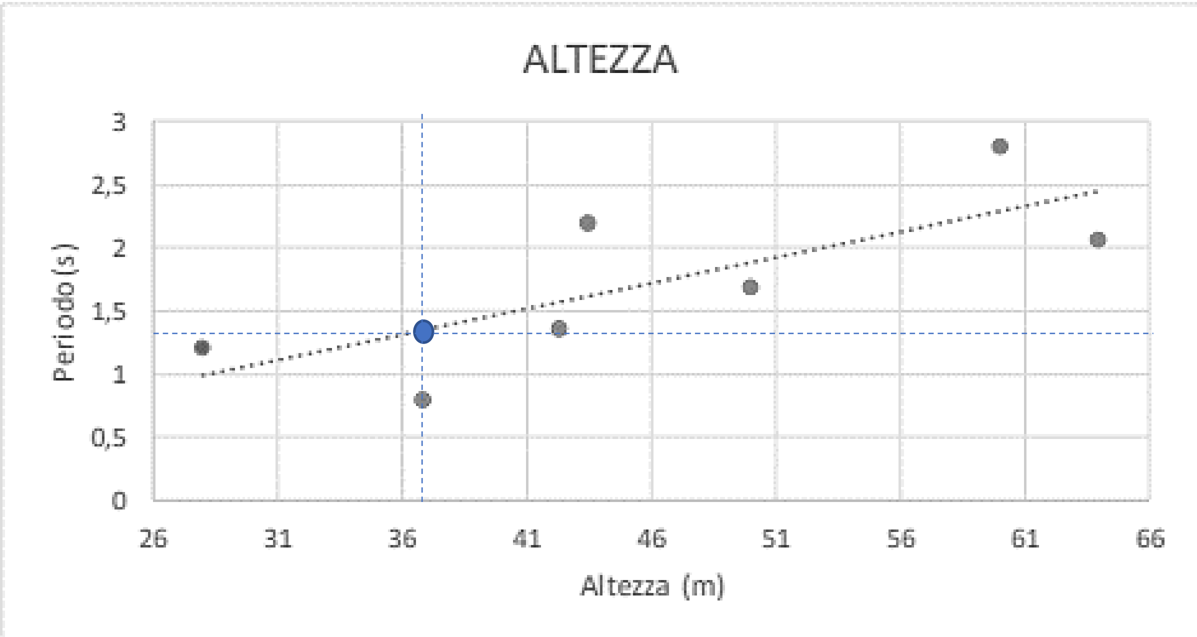
7.1.2.1 Comparison with other bell towers

It is useful to implement a comparison with past theses, to verify the veracity of the results obtained in terms of period, considering the height and the ratio of height to side of the steeple as useful parameters for comparison.

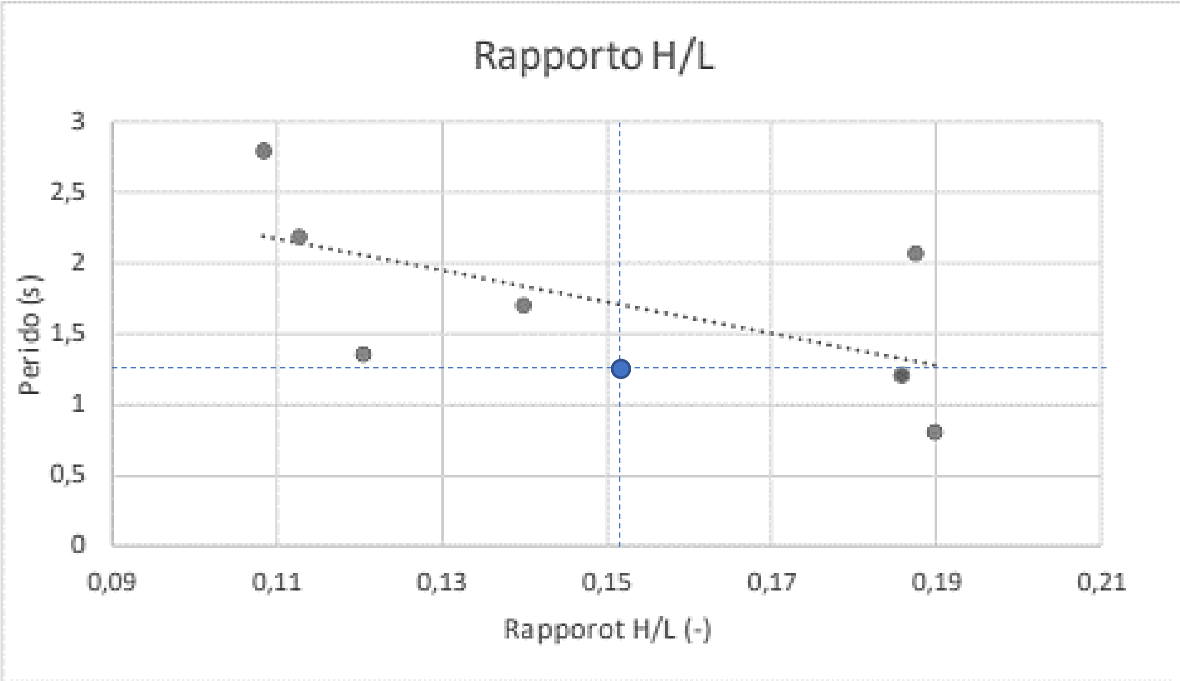
We then take up the thesis [23] to implement the comparison. The case study concerned the bell tower of the parish church of the Conversion of St. Paul, in Breda di Piave, Treviso, mentioning and taking into consideration also other bell towers previously studied.

Two summary tables are reported below to analyse whether the results obtained in this thesis are significant and reflect the typical trend of bell towers.

Table 7 - Comparison H-T (height and period) for different bell tower



The bell tower analysed follows the linear trend shown in the table.



The same is done in terms of H/L ratio, the bell tower of the case study is reported in blue.

7.1.3 Church (Structural Wall, Arches, Façade and Tower)

This analysis is carried out with the addition of the bell tower to understand how the behaviour of the structure changes with the addition of this very rigid element with a fairly high elevation. It was decided to add the belfry later because this, as described in the historical chapter, was added at a later stage than the construction of the church and therefore, also as noted during the analyses carried out in situ (thermography and sonic tests), it is not well connected with the rest of the structure and therefore is only supported but not well constrained with the façade and the other parts. It is also important to carry out this analysis because one can see the effect that the bell tower exerts on both its period and the period of the structure itself, analysed previously.

	Frequency	**Period [s]**
1	1,148	**0,871**
2	1,181	**0,847**
3	3,014	**0,332**
4	3,414	0,293
5	3,457	0,289
6	3,470	0,288
7	4,010	0,249
8	4,309	0,232
9	4,455	0,224
10	4,506	0,222
11	4,574	0,219
12	4,744	0,211
13	4,840	0,207
14	4,898	0,204
15	5,093	0,196
16	5,275	0,190
17	5,503	0,182
18	5,862	0,171
19	6,311	0,158
20	6,547	0,153

Table 8 - Modal Analysis, Structure & Tower

Mode 1

F [Hz]: 1.148

T [s]: 0.871

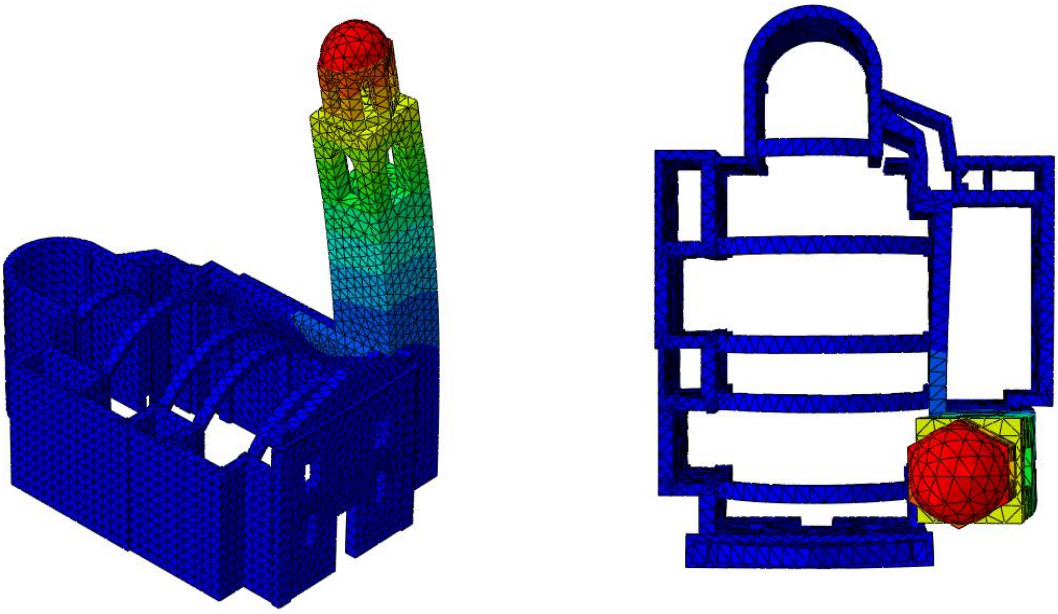


Figure 74 – Modal Analysis, Church - Bell Tower, Mode 1

Already in the first mode after the addition of the belfry, it can be seen how the initial damage mechanism would no longer concerns the façade but the top of the belfry; the rupture, in fact, resumes the first mode we presented for the single belfry.

It can be seen, however, how the period has really varied with respect to the analysis of the structure without the belfry and with respect to the period of the belfry itself; for the structure alone, in fact, the period of the first mode was 0.27s, while for the belfry alone it was 1.269s; here, on the other hand, the period is almost an average between the two previously described, equal to 0.87s.

This behaviour is reasonably due to the fact that initially the structure without the bell tower was much stiffer than the bell tower, given its low period; on the other hand, the bell tower, having a very high period, had less stiffness; by joining the two structures, the behaviour obviously changes, which means that the bell tower acquires stiffness by being joined to a second structure.

Mode 2

F [Hz]: 1.181

T [s]: 0.84

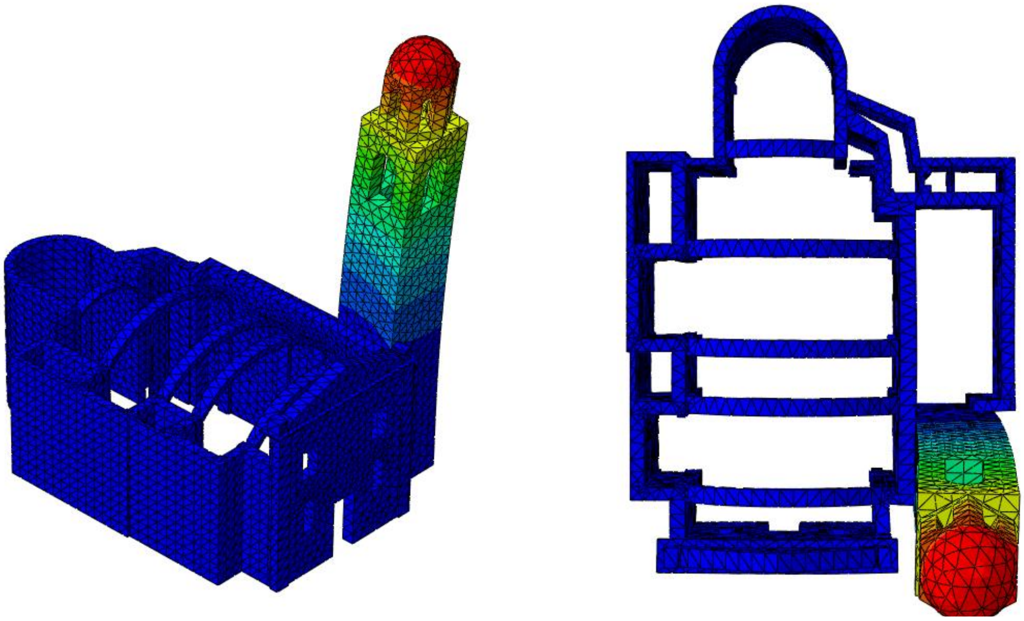


Figure 75 – Modal Analysis, Church - Bell Tower, Mode 2

The second mode also mainly concerns the steeple in its upper part, it is also reminiscent of the second mode of the single steeple, although the period is much shorter, due to the increased stiffness of the structure. While mode 3 below concerns the façade.

Mode 3

F [Hz]: 1.181

T [s]: 0.332

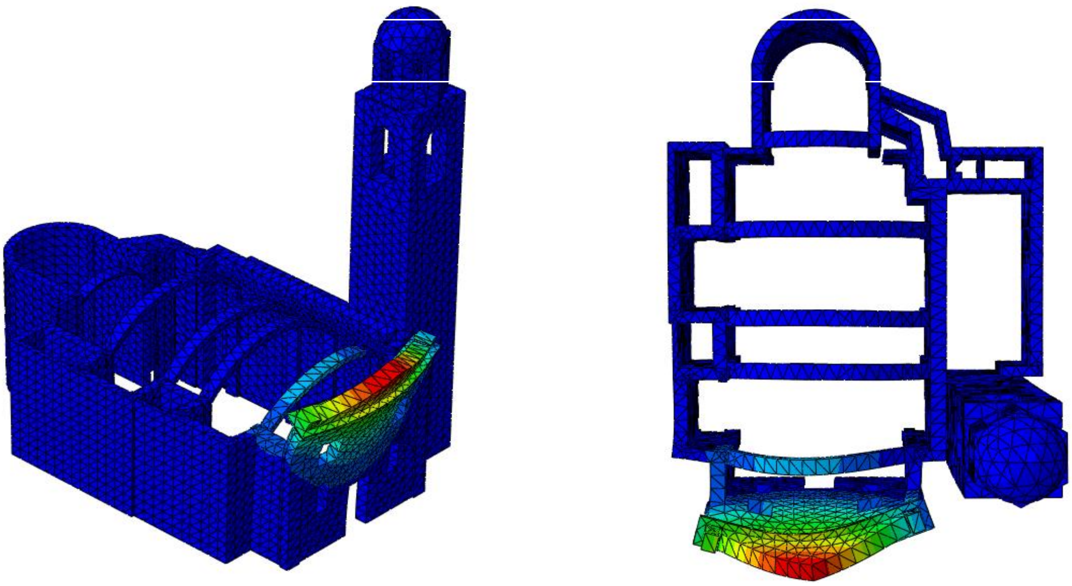


Figure 76 – Modal Analysis, Church - Bell Tower, Mode 3

7.1.4 Complete Model of the Church

After presenting the first modal analyses of the individual portions of the structure, joining them one at a time, to understand the variation in the global behaviour of the structure, the modal analyses of the complete structure are finally reported, where the vaults were then added. For simplification, only the vaults of the nave have been considered. In this case we report a comparison, for the first 20 modes, of all the periods based on the different cases reported. Also in this case, the first 10 modes are reported, the other are reported at the end of this thesis in the Appendix C.

	Bell Tower	***Wall, Façade & Tower***	**Complete Model**
	Period [s]	**Period [s]**	**Period [s]**
1	1,269	0,871	0,857
2	1,266	0,847	0,837
3	0,282	0,332	0,243
4	0,281	0,293	0,225
5	0,236	0,289	0,217
6	0,135	0,288	0,195
7	0,120	0,249	0,189
8	0,120	0,232	0,169
9	0,118	0,224	0,158
10	0,093	0,222	0,145
11	0,091	0,219	0,143
12	0,066	0,211	0,143
13	0,058	0,207	0,136
14	0,056	0,204	0,135
15	0,056	0,196	0,122
16	0,054	0,190	0,122
17	0,046	0,182	0,117
18	0,038	0,171	0,116
19	0,038	0,158	0,109
20	0,037	0,153	0,107

It can be seen how the import of the vaults reduces the period for each mode slightly; their presence is therefore not as fundamental as the rest of the structure, but it can be seen later how they change the behaviour of the entire nave, providing more stability and rigidity to the arches and the right wing.

Mode 1

T [s]: 0.857

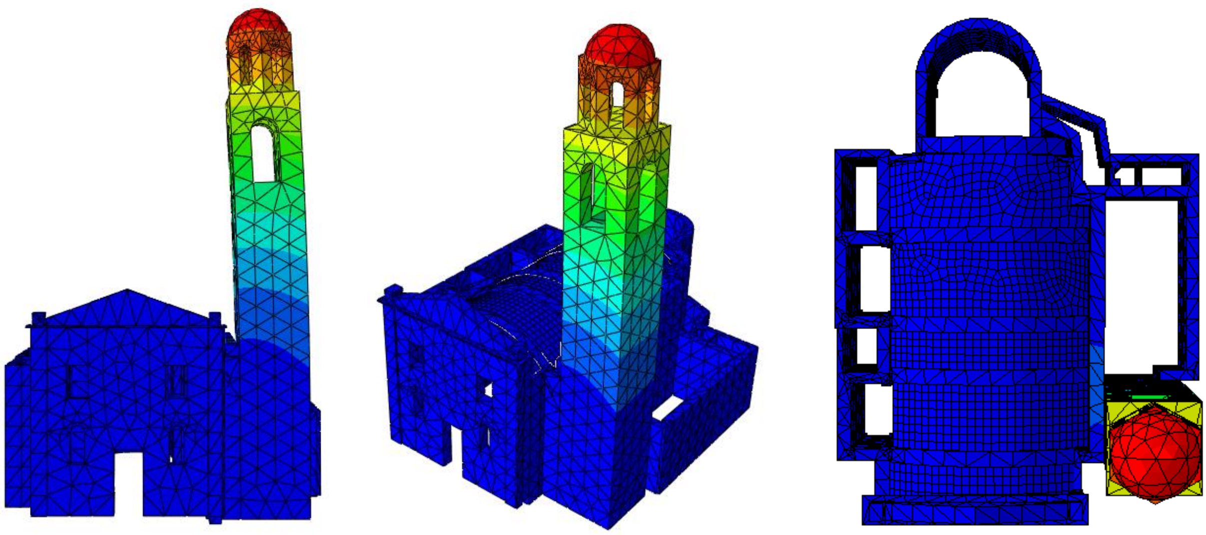


Figure 77 – Modal Analysis, Complete Model, Mode 1

Mode 2

T [s]: 0.837

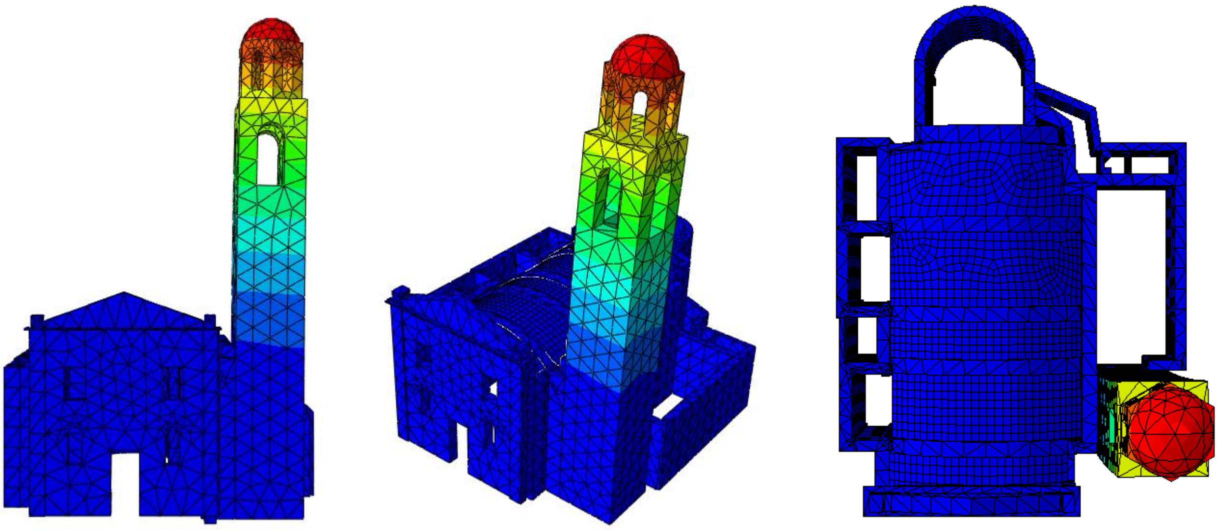


Figure 78 – Modal Analysis, Complete Model, Mode 2

Mode 3

T [s]: 0.243

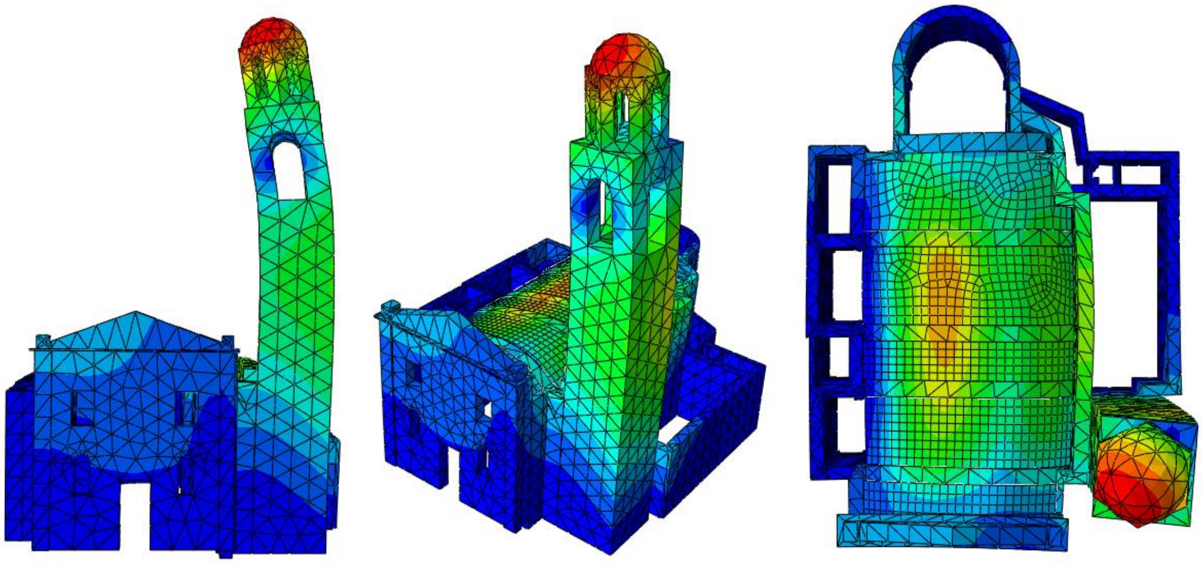


Figure 79 – Modal Analysis, Complete Model, Mode 3

Mode 4

T [s]: 0.225

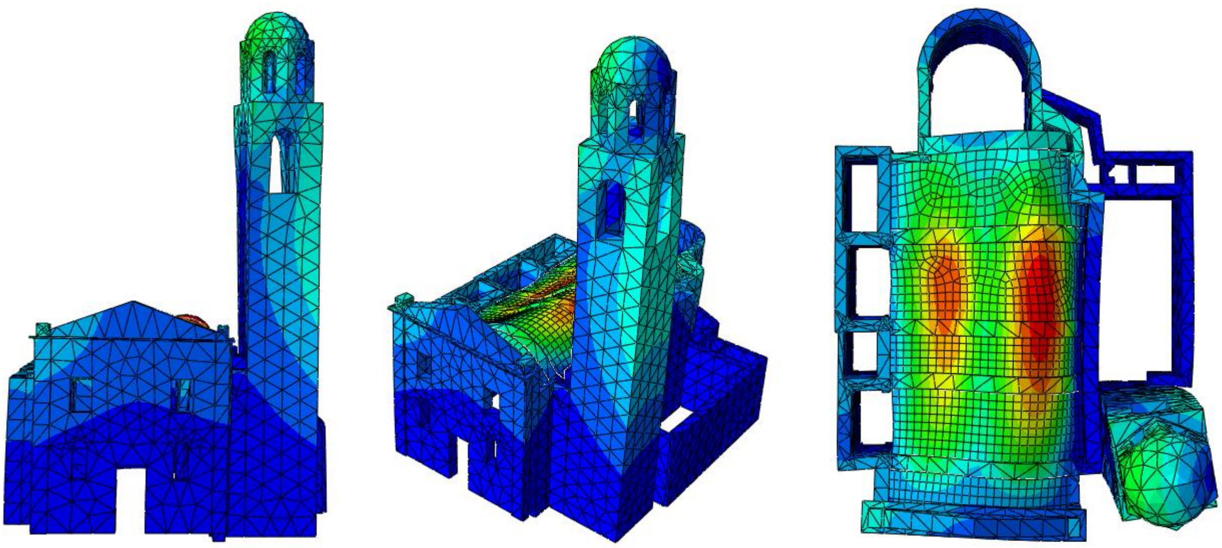


Figure 80 – Modal Analysis, Complete Model, Mode 4

Mode 5

T [s]: 0.217

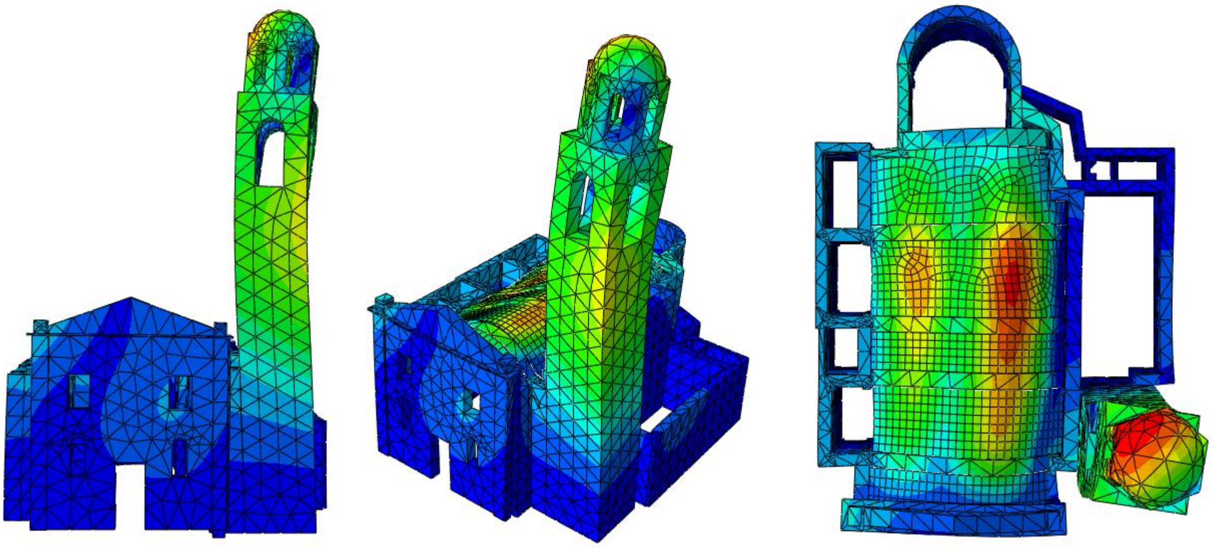


Figure 81 – Modal Analysis, Complete Model, Mode 5

Mode 6

T [s]: 0.195

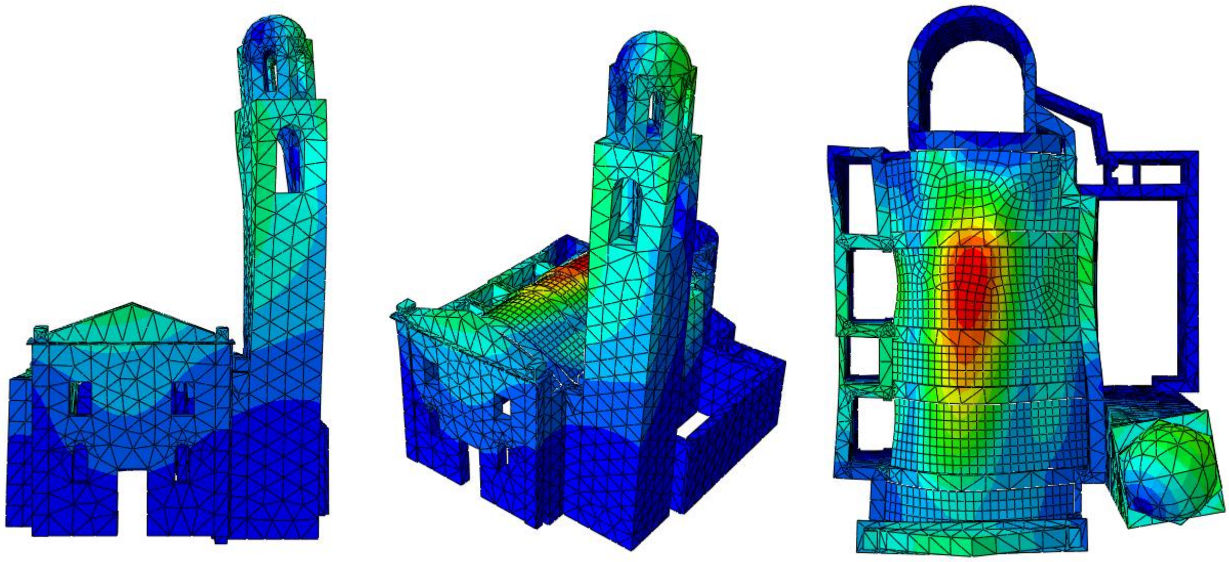


Figure 82 – Modal Analysis, Complete Model, Mode 6

Mode 7

T [s]: 0.189

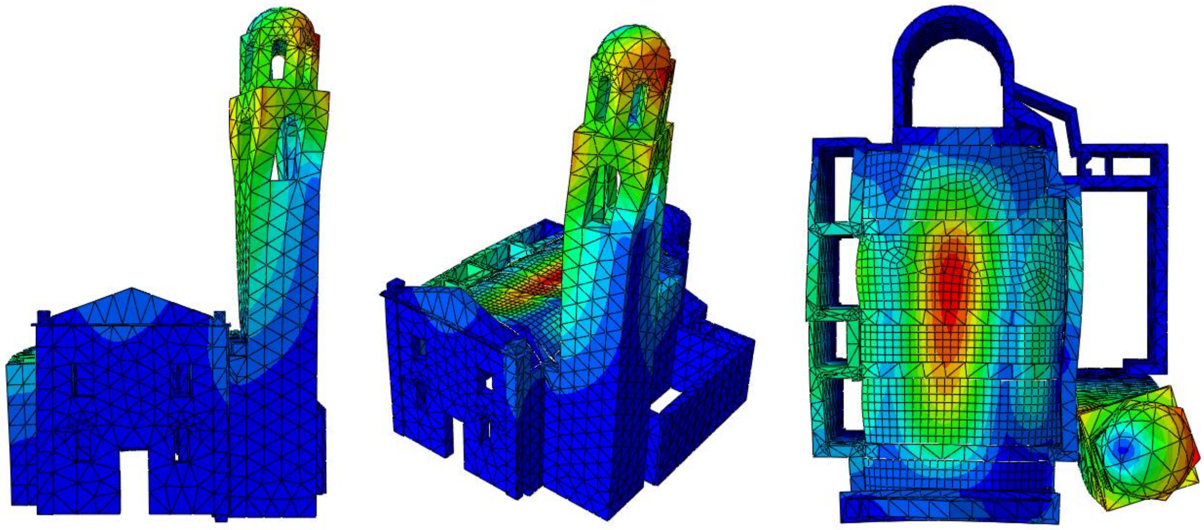


Figure 83 – Modal Analysis, Complete Model, Mode 7

Mode 8

T [s]: 0.169

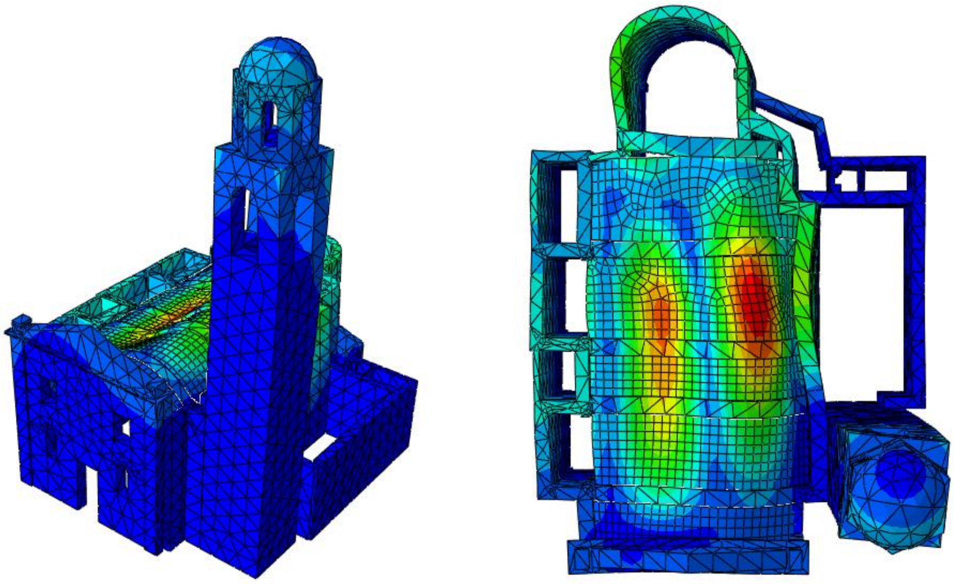


Figure 84 – Modal Analysis, Complete Model, Mode 8

Mode 9

T [s]: 0.158

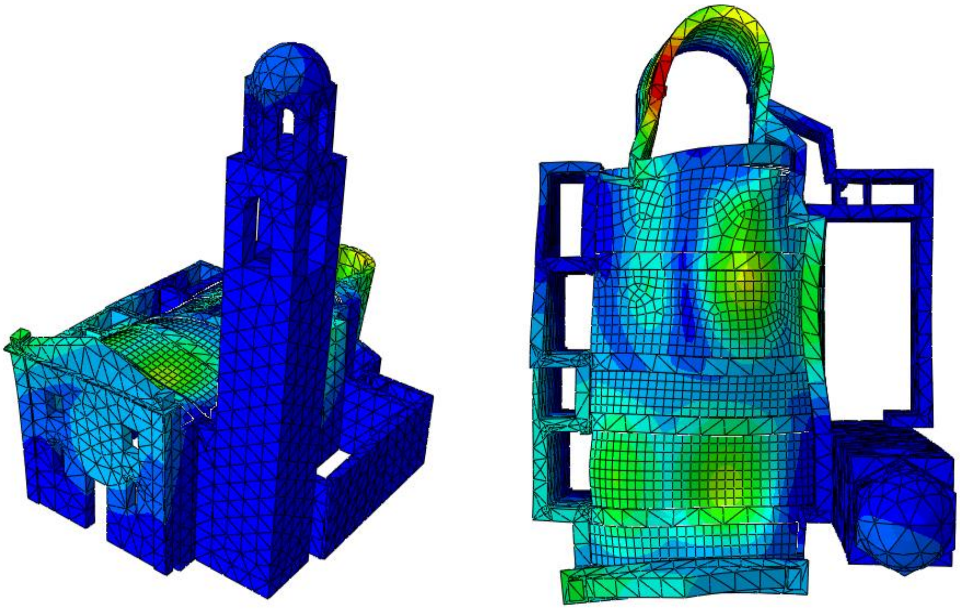


Figure 85 – Modal Analysis, Complete Model, Mode 9

Mode 10

T [s]: 0.145

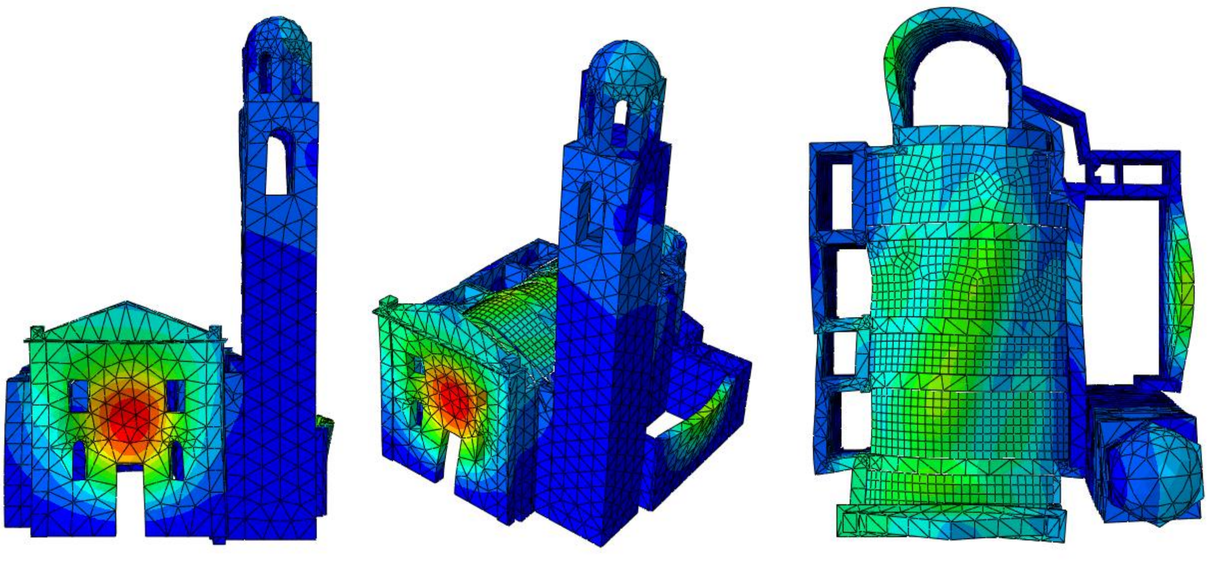


Figure 86 – Modal Analysis, Complete Model, Mode 10

7.2 Response Spectrum Analysis

The next step, done by inputting the response spectrum (shown and calculated in Chapter 4, of which only the acceleration values are given here) into Abaqus, allows us to evaluate the response of the structure under analysis according to the considered earthquake.

7.2.1 Church (Structural Wall, Arches and Façade)

The results for the response spectrum are reported initially only for the X direction:

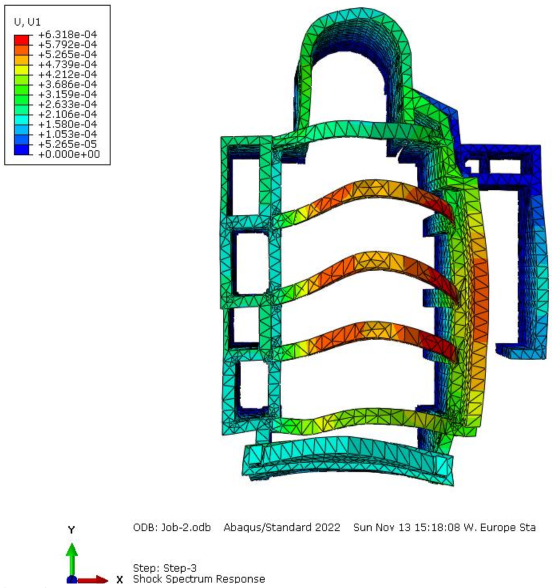


Figure 87 – Response Spectrum, Church

And for the Y direction:

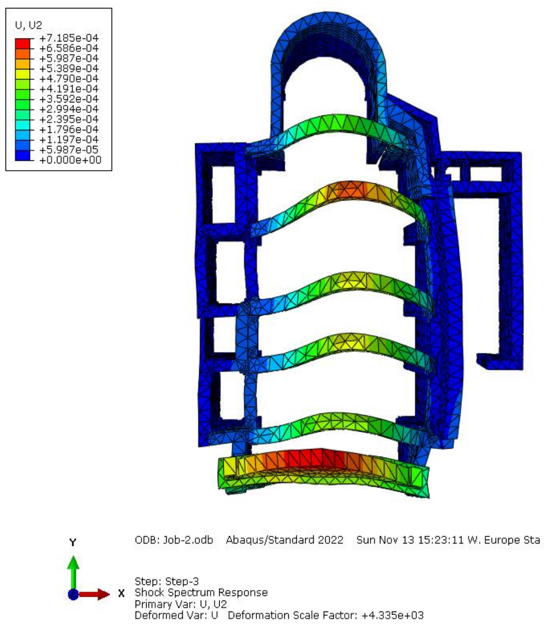


Figure 88 – Response Spectrum, Church

7.3 Results

For the verification of these parameters, the strength criteria are considered; materials can have different tensile and compressive behaviour in terms of strength and, therefore, an allowable tensile and compressive stress must be defined. Since we are dealing with a masonry building, the criterion to be considered is the Galileo-Rankine criterion, which identifies the maximum and minimum principal stresses as critical parameters of the stress state. Given a certain stress tensor S, it is possible to identify the reference from the principal directions where the tensor is diagonal:

$$S = \begin{bmatrix} \sigma_1 & 0 & 0 \\ 0 & \sigma_2 & 0 \\ 0 & 0 & \sigma_3 \end{bmatrix}$$

According to the criterion, it must be verified that:

$$-\sigma_C < \sigma_{(n)I-II} \leq \sigma_T$$

Where:

- σ_C is the value of the permissible stresses in simple compression.
- σ_T is the value of the permissible tensile stresses.

These, for this case study, are:

- $\sigma_C = 350 \frac{N}{cm^2}$, this value may be considered a good approximation considering the values of the masonry types table and the features and dimensions of the masonry walls.
- Since the tensile strength can be assumed approximately equal to $\frac{1}{10}$ of compression, and also, considering that this value is 5-6 times the shear resistance for masonry, that usually is around $6 - 9.2 \frac{N}{cm^2}$, it was decided to use a value that complied with approximately all the constraints reported, considering that $\sigma_T = 38 \frac{N}{cm^2}$

7.3.1 Bell Tower

For this verification, only the steeple and the complete model of the church are considered. The stress values, to be analysed, of the model considered are given below.

Positive tension is assumed.

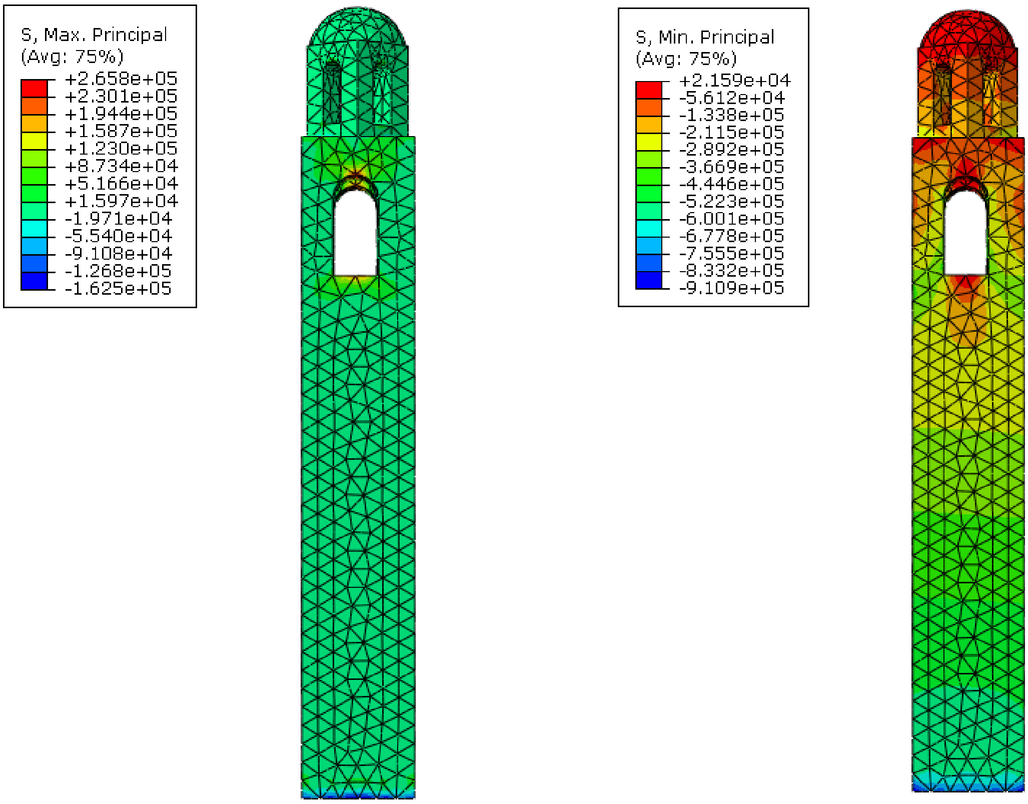


Figure 89 – S. Max and S. Min Principal, Bell Tower

From the above figures the maximum compression value is equal to $\sigma_{(I)} = -91.09 \frac{N}{cm^2}$:

$$\sigma_{(I)} = -91.09 \frac{N}{cm^2} > -350 \ \frac{N}{cm^2} \qquad OK$$

From the above figures the maximum tension value is equal to $\sigma_{(II)} = 26{,}58 \frac{N}{cm^2}$:

$$\sigma_{(II)} = 26{,}58 \frac{N}{cm^2} < 38 \ \frac{N}{cm^2} \qquad OK$$

For the bell tower, both tensile and compressive tests are verified, although the tensile value is almost close to the value beyond which damage to the structure occurs; the top has the most critical values.

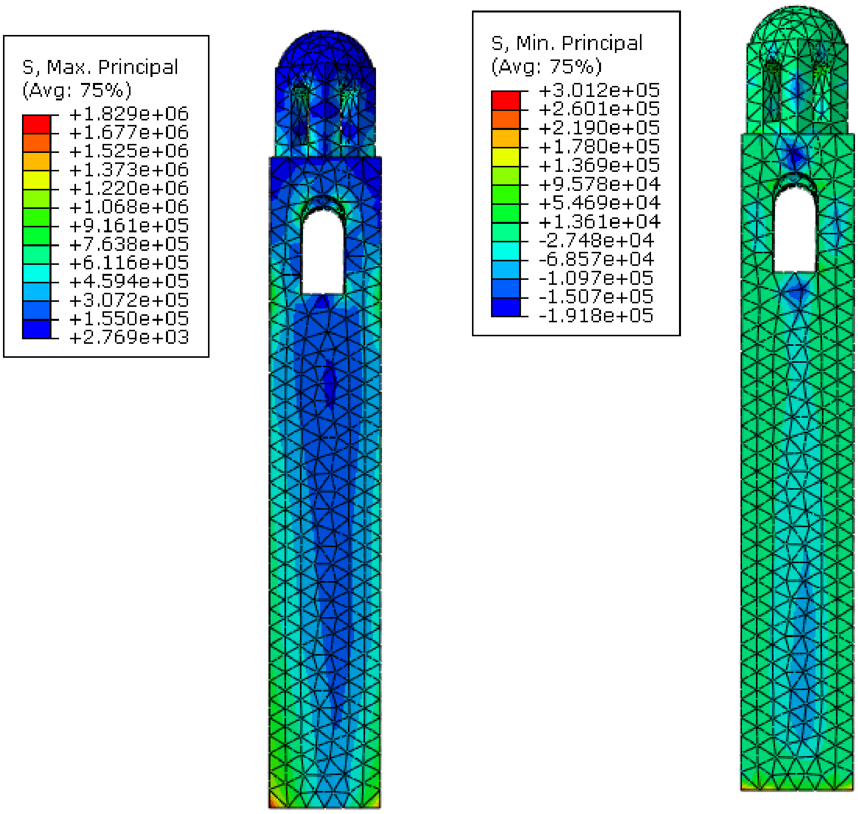


Figure 90 – S. Max and S. Min Principal, Bell Tower

The same is reported after the insertion of the response spectrum.

From the above figures the maximum compression value is equal to $\sigma_{(I)} = -182.9\frac{N}{cm^2}$:

$$\sigma_{(I)} = -182.9\frac{N}{cm^2} > -350\ \frac{N}{cm^2} \qquad OK$$

From the above figures the maximum compression value is equal to $\sigma_{(II)} = 19.18\frac{N}{cm^2}$:

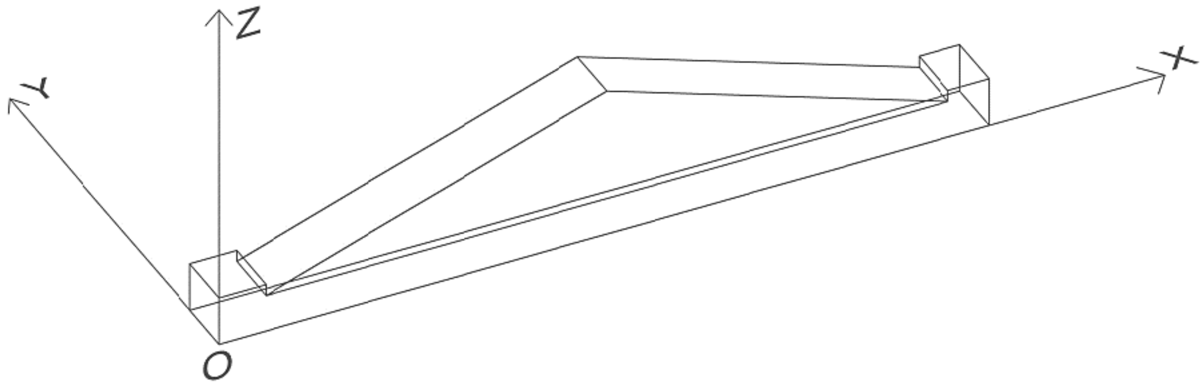
$$\sigma_{(II)} = 19.18\ \frac{N}{cm^2} < 38\ \frac{N}{cm^2} \qquad OK$$

The results are obtained considering both the static loads and the response spectrum.

8. LIMIT ANALYSIS

8.1 Linear cinematic analysis of the tympanum behaviour

Of all the damage mechanisms presented in the previous section, it was decided to proceed with a linear kinematic analysis of the gable of the façade, which is usually one of the main elements of interest in historic churches in the event of an earthquake.



Consideration must be given to the relationship that exists between the capacity, which represents the capacity and strength of a material, and the demand (real seismic action; the capacity must necessarily be greater than or equal to the demand in order to satisfy the verification. According to the NTC 2018, the collapse of the structure does not necessarily occur immediately after the mechanism has been activated, the demand is thus calculated:

$$Demand = \frac{Demand\ from\ the\ response\ spectrum}{q}$$

Where:

- "q", is the q-factor that consider the mechanism resistance, equal to 2.

For the evaluation of this fracture mechanism, several parameters must be considered, first the geometry of the tympanum itself, its volume, the behaviour factor and its unit weight. Based on the equilibrium method, it must be imposed that the resistant moment equals the acting moment, hence that $M_S = M_R$ in more detail:

$$a_0 M_S + M_{Ext} = M_{RC} + M_{RA} + M_{RF}$$

Where:

- M_S is the bending moment due to the actual seismic action.
- a_0 is the horizontal forces multiplier.
- M_{Ext} is the bending moment of the external actions, without the seismic one.
- M_{RC} is the bending moment, factor of the cohesive forces.
- M_{RA} is the bending moment, factor of the friction forces.
- M_{RF} is the bending moment, factor of the mechanism shape.

After the definition of all these parameters, also the activation acceleration $a_0 *$ must be defined, starting from the a_0 and considering the following formula:

$$a_0^* = \frac{\alpha_0 g}{e^* FC}$$

Where:

- g is the gravity, equal to 9,81 $\frac{m}{s^2}$
- FC is the confidence factor, described in the Chapter II.
- e^* is the participation mass of the mechanism.

$$e^* = \frac{M^* g}{\sum_{i=1}^{n+m} Pi}$$

Where:

- Pi is the gravitation forces.
- M^* is the participating mass (total mass of the system if applied in one point).

The verification depends on the ground level considered, in fact

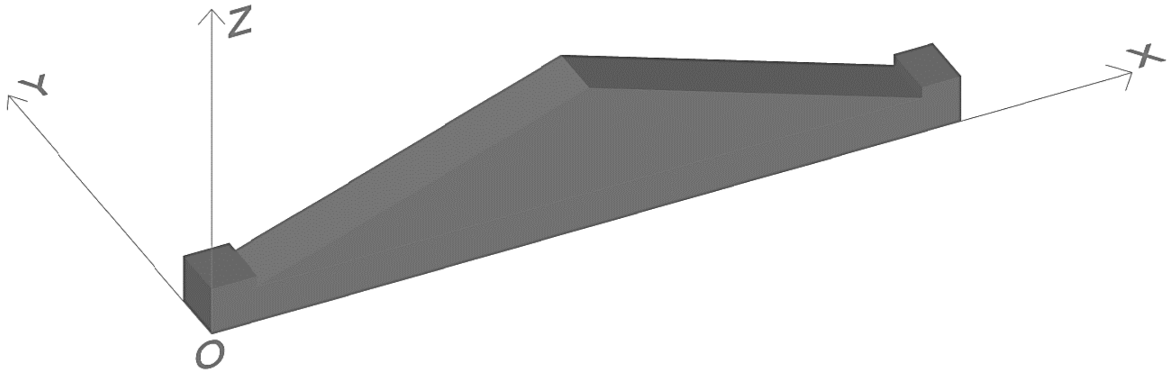
$$a_0^* \geq \frac{a_g(PVR)S}{q}, for\ the\ ground\ level$$

$$a_0^* \geq \frac{S_e(T1)\Psi(Z)\gamma}{q}, to\ a\ certain\ height\ from\ the\ ground$$

Where:

- $S_e(T1)$ is the acceleration in the response spectrum from the fundamental period of the considered structure.
- $\Psi(Z)$, is the first modal shape at a certain level of height Z.

The three-dimensional view of the subject and its dimensions are shown below, which is useful for continuing with the verification of the fracture mechanism.



In the table below, the main dimensions of the tympanum are shown, for the centre of gravity we refer to point G, the centre of gravity of the structure analysed.

Dimension	**Value**
Length (X)	13 m
Length (Y)	1 m
Height (Z)	2.65 m
Volume	20.69 m^3
Weight	18000 N/m^3
X_G (Centre of gravity)	6.5 m
Y_G (Centre of gravity)	0.5 m
Z_G (Centre of gravity)	0.88 m
Confidence Factor FC	1.12

After the definition of all these values, the α_0 is evaluated and is equal to $\alpha_0 = 0.568$

After the definition of all these parameters, also the activation acceleration a_0* must be defined, starting from the a_0 and considering the following formula:

$$a_0^* = \frac{\alpha_0 g}{e^* FC} = \frac{0.568 \cdot 9.81}{1 \cdot 1.12} = 4.976 \frac{m}{s^2} = 0.4976\, g$$

Now we must proceed with the verification of the gable, bearing in mind that it must be considered from ground level, as described above. Obviously, since we are considering the gable, we will have to consider the following formula, which concerns structures not at ground level, but placed at a certain height, in our case 12.71m.

The following formula must be used:

$$Demand = \frac{S_e(T1)\Psi(Z)\gamma}{q}$$

Where:

- $S_e(T1)$ is the acceleration from the fundamental period of the considered structure, and the period, in this case, is evaluated considering the codes and the normative, and so the period T1 is equal to:
$$T_1 = 0.05 \cdot H^{0.75} = 0.05 \cdot 12.71^{0.75} = 0.336\ s$$
Using the associated acceleration, the value of $S_e(T1) = 0.115\ g$
- $\Psi(Z)$ thanks to a simplification, is evaluated as $\frac{Z}{H} = \frac{13.56}{15.35} = 0.883$
- γ is the modal participation as 3N/(2N+1) where N is the number of storeys, and in our case is equal to 1.
- $q = 2$ is the parameter for the collapse mechanism.

Dimension	Value
S_e (T1)	0.115 g
Ψ(Z)	0.883
γ	1
q	2

$$Demand = \frac{S_e(T1)\Psi(Z)\gamma}{q} = \frac{0.115 \cdot 0.883 \cdot 1}{2} = 0.050\ g$$

$$a_0^* = 0.4976\ g \geq 0.050\ g$$

The mechanism is verified and will not be activated with the response spectrum entered.

9. CONCLUSIONS

The aim of the thesis is to analyse, from a seismic point of view, the damage that could affect the historical building under consideration; this was carried out through finite element simulations using the structural analysis software Abaqus.

The case study concerns a church in the Lodigiano area, the Church of the Assumption of the Blessed Virgin Mary in Tavazzano con Villavesco (LO). for which the construction history was described, in terms of original layout and of the main restorations carried out, and compared with the current state of conservation. Fortunately, the church in question has not suffered significant damage in past earthquakes; despite this, an extended crack pattern is presented, especially about the bell tower. The bell tower itself has been the subject of specific analyses, to assess its behaviour as an element, and of overall analyses, to verify its behaviour within the historic church building. As far as the rest of the church is concerned, the main damage concerns the detachment of plaster from the walls and ceiling, and the numerous cracks that sometimes extend to the full height.

Initially, prior to the analyses with the finite element modelling software, some non-destructive tests were carried out for the church, to assess the mechanical properties of the materials, the state of preservation and to make a comparison, in terms of geometry and thickness, with the documentation provided by the Diocese of Lodi.

The first investigations carried out were thermographic surveys, useful for analysing the masonry structure; from these analyses it emerged that the masonry is heterogeneous, the results are in fact not excellent. The areas tested show very different velocities, this is due to poor bonding and the use of different materials, due to the construction in different historical periods; in fact, as explained in the historical chapter, the church has undergone many changes, moving the bell tower, plugging some openings that are now closed, and even adding some volumes. With subsequent analyses, thanks to sonic tests, it was also possible to assess the internal state of conservation. In fact, many cracks are present due to the detachment of the plaster from the masonry apparatus; this plaster detachment that causes the creation of air-filled cavities worsens the state of conservation of the church itself. As also mentioned in the historical chapter, this structure is in a very humid location, within the Po Valley, and therefore externally there are also phenomena of capillary rising that have degraded much of the external surface.

Also, thanks to the performance of these tests, it was possible to assume that the apse was built at a different time than the rest of the load-bearing structure, in fact here the thermographs do not give uniform results, but rather uneven and different with respect to the rest of the structure. Another fundamental element, needed in order to carry out the subsequent seismic analyses, was the bell tower, which, having been built later than the main structure of the church, may be expected to be only connected to the façade and the right wing, which is the reason for the individual analyses carried out on the bell tower alone. Other wall areas, on the other hand, identify that the structure is composed not only of one, but of several leaves, which, however, compared to the main façade, are very well-balanced. The last analysis carried out in situ the sonic test to measure the thickness of the walls, was very useful for comparing the geometric measurements with the dimensions provided by the Diocese of Lodi, which sent us the documents. The on-site measurements showed how the stratigraphy of the walls are quite different in terms of thickness, in fact the main façade, after the sonic test, reported much higher values than the documents quoted. These measurements were useful to understand the geometric configuration of the construction and then to be able to better model the structure for subsequent analysis.

The subsequent analyses were conducted after the creation of the architectural model, created with the parametric and modelling software Rhino; this model was made as realistic as possible, also in terms of textures and graphics, to best guarantee the reader's understanding of the structure. From the architectural model, the structural model was then modelled, the one used for the analyses; obviously this step involved considerable simplifications for all parts of the work.

The decorative architectural parts have not been considered, the small recesses, often a few centimetres, have been made regular to make the analysis as simple as possible. For modelling purposes, the load-bearing structure of the façade and the arches remained very similar, almost identical to the architectural model, the vaults on the other hand were considerably simplified, this is because the rich presence of cross vaults and barrel vaults made it difficult to reproduce them in the model, for this reason the main vaults were considered, that is, those running along the nave and covering the apse. Even the roof, modelled accurately in the architectural model in the structural model, was then replaced and considered as a single load: this was done to simplify the import of the blocks and to simplify an already very detailed simulation with a high numbers of finite elements.

The first analysis was a gravity load analysis performed only on the main elements of the structure, to investigate the initial results and the behaviour of the structure only under its own weight. Subsequently, it was decided to proceed with modal analyses, carried out on three different models, because we wanted to verify how the individual elements could modify the global behaviour of the structure. Initially, therefore, the belfry and vaults were not considered, for the reasons mentioned above; the belfry was then first analysed individually and subsequently as an additional element joined to the structure of the church, and finally the complete model was analysed.

These three different simulations were able to demonstrate how each element of the structure has a specific task and a considerable importance with respect to the overall behaviour. In fact, if initially the first modal shape, and probably the first damage mechanism, in the absence of the bell tower, concerned the façade, after the addition of the bell tower, the structure showed initial modes that concerned only the bell tower. Initially, the period of the structure was very low, around 0.27s, and that of the belfry was 1.27s, after the union of the two parts, the overall period turned out to be an average, approximately, of the two behaviours of the structures taken individually. In fact, the bell tower was not very rigid, having a high period, and vice versa, the supporting structure was very rigid. The importation of the vaults also played a key role on the deformations suffered by the arches before their insertion. In fact, they provided more rigidity to the arches themselves, which were initially too deformable, but did not contribute decisively to the period variation; their presence turns out to be decisive because they influence the displacements from the façade structure; without vaults, the stress distribution at these points was very high.

It is interesting to observe the mode shift once the model passes from that of the church alone to the one that includes the bell tower and finally when the vault over the nave is added. Initially, the first modal shape concerns mainly the façade, making its tip, the tympanum, oscillate. This mode has a relatively short period, 0.27 seconds.

The analysis of the complete model shows that the belfry is the most stressed element; in fact, the first two modes of vibration are identical to the two first modes of the single belfry, and the period is extremely high, compared to the periods found from the third mode onwards, which only affect the important central structure.

Other elements that present high deformability from modal analysis and so may be subjected to major damage concern the façade, for which the rupture mechanism of the tympanum was specifically studied, the arches and the vaults.

The same first mode that entails the tympanum in the first model, with only the church, involves, as well, the lateral nave wall on the right-hand side, which is particularly deformable, not being retained for a long length, while the wall at left is stiffened by the continuous presence on the lateral chapels. Once the bell tower is inserted into the model, the first mode effectively affects only the bell tower itself, reproducing only the first modal shape of the tower and moving the façade mode to the third position. As already observed and mentioned, the bell tower oscillates now with shorter periods, due to the increased stiffness at the base from the church walls and to the reduced free elevation.

The façade mode, however, has also changed its characteristics: the mode involves only the façade as the lateral nave wall is stiffened by the presence of the bell tower. In the third configuration, in which the vaults have also been inserted, the tympanum results retained by the vault. A lateral rotation of tympanum appears in a much higher mode and only mildly as a secondary effect, while the main deformation is in the vault itself.

The vault appears then to act as a protective element for the tympanum, an element most often damaged in earthquakes. For such protective effect, however, the connection must be effective. Yet very likely the two elements have been built separately so the continuity is not guaranteed. Probably the actual situation is intermediate, which would make some further investigation interesting. Similarly, in the second and third model the bell tower has been developed as part of the church. Yet, the different construction period and the consequent low intersection of the walls of the bell tower and of the church may not produce such effect. Examining independent behaviors of church and belltower, as well as the consolidated one, allows to describe the limits of the real situation.

All the three models are interesting. This is because, when combined, they allow us to trace the limits of the real behavior of the church compound.

A more punctual investigation at the level of connection would be required to assess the actual situation of the church more accurately in this area.

The last analyses conducted concerns the failure mechanism of the façade tympanum by means of the limit analysis method. Considering all the parameters described in the code, it was possible to demonstrate that the collapse of the gable should not occur, as the demand resulted much lower than its capacity.

Obviously, it must be borne in mind that the study depends on the choice of materials made during the development of the structural model, of which we had no certain and strong information, but only non-destructive analyses carried out in situ; it also obviously depends on the geometry, the constraints and the degree of interlocking between the various components of the structure. It can be said that the model presents a realistic behaviour, obviously the choice of proceeding with a linear analysis, which is capable to simulate the structure only up to the elastic limit, is not useful to further investigate the condition of the structure in more detail; in order to have a complete vision one should in fact proceed with complete dynamic and non-linear analyses, after a mre extended diagnostic campaign.

In a continuation of this study, it would be appropriate to examine small portions of the structure where non-destructive testing has revealed that the degree of bonding and interlocking between the different layers is not good, in order to try to understand the causes of this condition and possibly to plan future repairs and restoration on the basis of more in-depth and specific results.

BIBLIOGRAPHY

[1]. D.M. 17/1/2018 – Aggiornamento delle «*Norme tecniche per le costruzioni*»

[2]. Abaqus Analysis User's Guide.

[3]. Abaqus/CAE User's Manual.

[4]. Circolare 21/1/2019, n.7 Consiglio Superiore LL.PP. – *Istruzioni per l'applicazione dell'"Aggiornamento delle «Norme tecniche per le costruzioni»*" of D.M 17/1/2018

[5]. L. Cantini, M.A. Parisi; "Cultural heritage and seismic risk evaluation. A critical review on the estimation of the safety level for historical buildings"; *Proceedings, 10th International Masonry Conference*, editors G. Milani, A. Taliercio and S. Garrity; Milan, Italy, July 9-11, 2018.

[6]. Tateo, V.; Sferrazza Papa, G. Seismic vulnerability of churches. First results of the study on three façade typologies, *Atti Convegno ANIDIS*, 2019.

[7]. EN 1998-1 (2004) (English): Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions, and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC].

[8]. CNR-DT 207 R1/2018 – Istruzioni per la valutazione delle azioni e degli effetti del vento sulle costruzioni.

[9]. EN 1992-1-1 (2004) (English): Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC].

[10]. DPCM 9 febbraio 2011, Valutazione e riduzione del rischio sismico del patrimonio culturale con riferimento alle norme tecniche per le costruzioni di cui al decreto del Ministero delle Infrastrutture e dei trasporti del 14 gennaio 2008, G.U., 2011.

[11]. M. Locatelli, «Analisi sismica della struttura della chiesa di San Domenico in Cagliari,» Tesi di Laurea Magistrale, Politecnico di Milano, 2014.

[12]. Bosiljkov, Vlatko, Mojmir Uranjek, Roko Žarnic, and Violeta Bokan-Bosiljkov; "An Integrated Diagnostic Approach for the Assessment of Historic Masonry Structures"; *Journal of Cultural Heritage*, Volume 11 Issue 3, July-September 2010, pp. 239-249; 2010.

[13]. M. Valente, G. Barbieri e L. Biolzi , «Seismic assessment of two masonry Baroque churches damaged by the 2012 Emilia earthquake,» *Engineering Failure Analysis,* vol. 79, pp. 773-802, 2017.

[14]. I. Grave, "Diagnostica e conservazione delle murature storiche: Studio preparatorio per il restauro conservativo della Chiesa dell'Assunzione della Beata Vergine Maria a Tavazzano con Villavesco", Tesi di Laurea, Politecnico di Milano, 2022.

[15]. F. Gravante, "Structural assessment for Cultural Heritage: the church of S. Pietro in Casolate", Tesi di Laurea Magistrale, Politecnico di Milano, 2019.

[16]. J. Gaeli, E. Marelli, "Analisi dei danni nelle chiese soggette a sisma: chiesa di s. Giovanni battista in s. Giovanni del dosso", Master Thesis, Politecnico di Milano.

[17]. Sferrazza Papa. G.; Silva. B. "Assessment of Post-Earthquake Damage: St. Salvatore Church in Acquapagana, Central Italy", *Buildings* 2018, *8*(3).

[18]. Bernardini. A.; Lagomarsino. S. The seismic vulnerability of architectural heritage. *Structures & Buildings*, January 2008.

[19]. G. Torelli, D. D'Ayala, M. Betti e G. Bartoli, «Analytical and numerical seismic assessment of heritage masonry towers» Bulletin of Earthquake Engineering, 18, pages 969–1008, 2020.

[20]. A. Kappos, "Design of earthquake resistant buildings". *Izgradnja*, V. 64, no. 5-6, 2010, 305-344", 2010.

[21]. E. Devetak, "Earthquake vulnerability assessment of a masonry monument including soil-foundation structure interaction", Tesi di Laurea Magistrale, Politecnico di Milano, 2018-2019.

[22]. R. Fleres, “La vulnerabilità sismica degli edifici monumentali la chiesa normanna di Santa Maria della Valle a Messina”, Tesi di Laurea Magistrale, Politecnico di Milano, 2016.

FORM A-DC

Presidenza del Consiglio dei Ministri
DIPARTIMENTO DELLA PROTEZIONE CIVILE

Ministero
per i Beni e le Attività Culturali

GRUPPO DI LAVORO PER LA SALVAGUARDIA E LA PREVENZIONE DEI BENI CULTURALI DAI RISCHI NATURALI

SISMA

EMERGENZA POST-SISMA

SCHEDA PER IL RILIEVO DEL DANNO AI BENI CULTURALI – CHIESE

MODELLO A – DC
Prima sezione

A_1

Data ⌴⌴ ⌴⌴ ⌴⌴⌴⌴	N° progressivo ⌴⌴⌴	N° Scheda ⌴⌴⌴⌴⌴⌴ *(a cura dell'ufficio)*

A_2 – RIFERIMENTO VERTICALE

Bene complesso ❍		Bene individuo ❍	
Denominazione bene complesso:			
Numero schede beni componenti ⌴⌴		Codice livello superiore	
Tipologia	❑ chiesa ❑ canonica ❑ palazzo ❑ castello ❑ torre ❑ bene archeologico ❑ altro		
Pianta	❍ regolare ❍ con cortili ❍ ad ali aperte ❍ lineare ❍ altro		

A_3 – LOCALIZZAZIONE GEOGRAFICO AMMINISTRATIVA

Regione	Codice Istat comune	*Indirizzo*
Provincia		1 ❍ via
Comune		2 ❍ corso
		3 ❍ vicolo
Località		4 ❍ piazza
		5 ❍ località num.civico
Sezione censuaria	N° complesso o aggregato	N° edificio
Foglio	Data ⌴⌴ ⌴⌴ ⌴⌴⌴⌴	Particelle	Sub.

A_4 – COORDINATE UTM

Quadrante	Longitudine Est (x) ⌴⌴° ⌴⌴'	Latitudine Nord(y) ⌴⌴° ⌴⌴'	❍ Lettura GPS

A_5 – OGGETTO

Denominazione bene:	
Denominazione storica:	
Datazione: anno secolo epoca Ultima trasformazione	
Proprietà:	☎
Utilizzatore:	☎

A6 – DESTINAZIONE D'USO ATTUALE

Uso		Utilizzazione temporale			Affollamento
		Continuo	Saltuario	Non utilizzato	
Cattedrale / Duomo	❑	❍	❍	❍	❑
Chiesa parrocchiale	❑	❍	❍	❍	❑
Oratorio	❑	❍	❍	❍	❑
Santuario	❑	❍	❍	❍	❑
Museo	❑	❍	❍	❍	❑
Auditorium	❑	❍	❍	❍	❑
Servizi	❑	❍	❍	❍	❑
Altro ______________	❑	❍	❍	❍	❑

A7 – CARATTERISTICHE DEL SITO

In piano ❍	Su rilievo / su cresta / su vetta ❍	Su riporto ❍	In pendio / su versante ❍	Avvallamento ❍

A8 – CONTESTO URBANO E POSIZIONE

Centro urbano ❍	Periferia urbana ❍	Area industriale - commerciale ❍	Area agricola ❍	Centro storico ❍

Isolata ❍	Connessa con altri edifici ❍ su __ lati	Altro ❍ ....

A9 – INFRASTRUTTURE

Accesso pedonale	❍	Rete viaria idonea in relazione al rischio	❑
Accesso carrabile	❍	Parcheggio nelle vicinanze	❑
Accesso con altezza inferiore a 4 metri	❍	Spazi aperti a disposizione	❑
Accesso con mezzi pesanti	❍	Altro ❑ _________________	

A10 – PRESENZA DI RISCHIO

		RILEVAZIONE DIRETTA	INFORMAZIONI ACQUISITE
Insediamento minacciato da frana	❑	❍	❍
Insediamento in zona alluvionabile	❑	❍	❍
Insediamento soggetto a minacce di tipo industriale	❑	❍	❍
Insediamento soggetto ad altre minacce naturali	❑	❍	❍

A11 – TIPOLOGIA DEI BENI ARTISTICI PRESENTI

TIPOLOGIA		Num.	superficie	TIPOLOGIA		Num.	superficie
Affreschi	❑	__	___	Dipinti mobili su vario supporto	❑	__	___
Mosaici	❑	__	___	Arredi (soffitti, amboni, pulpito, stalli corali)	❑	__	___
Stucchi	❑	__	___	Decorazioni plastiche mobili	❑	__	___
Arazzi	❑	__	___	Manufatti in carta e pergamena	❑	__	___
Altari / statue	❑	__	___	Reperti archeologici	❑	__	___
Libri / Stampe	❑	__	___	Altri	❑	__	___

A12 – DOCUMENTAZIONE FOTOGRAFICA - Realizzata da .... SI ❍ NO ❍

A13 – COMPILATORE SCHEDA

Cognome __________________	Nome ______________
Ente/ufficio di appartenenza ______________________________	
☎ ___________	E-Mail: ....

A14 - RIFERIMENTO SCHEDA DELLA VULNERABILITA' DELLE CHIESE

N° Scheda	Data	Ente

A15 - STATO DI MANUTENZIONE GENERALE

Buono ❍	Discreto ❍	Scadente ❍	Pessimo ❍	In corso lavori ❑
Eventuali precedenti lesioni esistenti	NO ❍	SI ❍	Limitate ❍ Estese ❍	Gravi ❍

A16 - DANNO SISMICO (Abaco dei meccanismi di collasso delle chiese)

LIVELLO DI DANNO

0 - □□□□□ assenza di danno
1 - ■□□□□ danno lieve
2 - ■■□□□ danno moderato
3 - ■■■□□ danno grave
4 - ■■■■□ danno molto grave
5 - ■■■■■ crollo

IDENTIFICAZIONE DEL DANNO

❑ danno sismico
❑ danno pregresso
❑ aggravamento

1	**RIBALTAMENTO DELLA FACCIATA**	❑
danno	DISTACCO DELLA FACCIATA DALLE PARETI O EVIDENTI FUORI PIOMBO	❑❑❑❑❑
2	**MECCANISMI NELLA SOMMITÀ DELLA FACCIATA**	❑
danno	RIBALTAMENTO DEL TIMPANO, CON LESIONE ORIZZONTALE O A V – DISGREGAZIONE DELLA MURATURA O SCORRIMENTO DEL CORDOLO – ROTAZIONE DELLE CAPRIATE	❑❑❑❑❑
3	**MECCANISMI NEL PIANO DELLA FACCIATA**	❑
danno	LESIONI INCLINATE (TAGLIO) – LESIONI VERTICALI O ARCUATE (ROTAZIONE) – ALTRE FESSURAZIONI O SPANCIAMENTI	❑❑❑❑❑
4	**PROTIRO – NARTECE**	❑
danno	LESIONI NEGLI ARCHI O NELLA TRABEAZIONE PER ROTAZIONE DELLE COLONNE – DISTACCO DALLA FACCIATA – MARTELLAMENTO	❑❑❑❑❑
5	**RISPOSTA TRASVERSALE DELL'AULA**	❑
danno	LESIONI NEGLI ARCONI (CON EVENTUALE PROSECUZIONE NELLA VOLTA) – ROTAZIONI DELLE PARETI LATERALI – LESIONI A TAGLIO NELLE VOLTE – FUORI PIOMBO E SCHIACCIAMENTO NELLE COLONNE	❑❑❑❑❑
6	**MECCANISMI DI TAGLIO NELLE PARETI LATERALI (RISPOSTA LONGITUDINALE)**	❑
danno	LESIONI INCLINATE (SINGOLE O INCROCIATE) – LESIONI IN CORRISPONDENZA DI DISCONTINUITÀ NELLA MURATURA	❑❑❑❑❑
7	**RISPOSTA LONGITUDINALE DEL COLONNATO NELLE CHIESE A PIÙ NAVATE**	❑
danno	LESIONI NEGLI ARCHI O NEGLI ARCHITRAVI LONGITUDINALI – SCHIACCIAMENTO E/O LESIONI ALLA BASE DEI PILASTRI – LESIONI A TAGLIO NELLE VOLTE DELLE NAVATE LATERALI	❑❑❑❑❑
8	**VOLTE DELLA NAVATA CENTRALE**	❑
danno	LESIONI NELLE VOLTE DELL'AULA CENTRALE – SCONNESSIONI DELLE VOLTE DAGLI ARCONI	❑❑❑❑❑
9	**VOLTE DELLE NAVATE LATERALI**	❑
danno	LESIONI NELLE VOLTE O SCONNESSIONI DAGLI ARCONI O DALLE PARETI LATERALI	❑❑❑❑❑
10	**RIBALTAMENTO DELLE PARETI DI ESTREMITÀ DEL TRANSETTO**	❑
danno	DISTACCO DELLA PARETE FRONTALE DALLE PARETI LATERALI – RIBALTAMENTO O DISGREGAZIONI DEL TIMPANO IN SOMMITÀ	❑❑❑❑❑
11	**MECCANISMI DI TAGLIO NELLE PARETI LATERALI DEL TRANSETTO**	❑
danno	LESIONI INCLINATE (SINGOLE O INCROCIATE) – LESIONI ATTRAVERSO DISCONTINUITÀ	❑❑❑❑❑
12	**VOLTE DEL TRANSETTO**	❑
danno	LESIONI NELLE VOLTE O SCONNESSIONI DAGLI ARCONI E DALLE PARETI LATERALI	❑❑❑❑❑
13	**ARCHI TRIONFALI**	❑
danno	LESIONI NELL'ARCO – SCORRIMENTO DI CONCI – SCHIACCIAMENTO O LESIONI ORIZZONTALI ALLA BASE DEI PIEDRITTI	❑❑❑❑❑

14	**CUPOLA – TAMBURO/TIBURIO**	❑
danno	LESIONI NELLA CUPOLA (AD ARCO) CON EVENTUALE PROSECUZIONE NEL TAMBURO	❑❑❑❑❑
15	**LANTERNA**	❑
danno	LESIONI NEL CUPOLINO DELLA LANTERNA – ROTAZIONI O SCORRIMENTI DEI PIEDRITTI	❑❑❑❑❑
16	**RIBALTAMENTO DELL'ABSIDE**	❑
danno	LESIONI VERTICALI O ARCUATE NELLE PARETI DELL'ABSIDE – LESIONI VERTICALI NEGLI ABSIDI POLIGONALI – LESIONE AD U NEGLI ABSIDI SEMICIRCOLARI	❑❑❑❑❑
17	**MECCANISMI DI TAGLIO NEL PRESBITERIO O NELL'ABSIDE**	❑
danno	LESIONI INCLINATE (SINGOLE O INCROCIATE) – LESIONI IN CORRISPONDENZA DI DISCONTINUITÀ MURARIE	❑❑❑❑❑
18	**VOLTE DEL PRESBITERIO O DELL'ABSIDE**	❑
danno	LESIONI NELLE VOLTE O SCONNESSIONI DAGLI ARCONI O DALLE PARETI LATERALI	❑❑❑❑❑
19	**MECCANISMI NEGLI ELEMENTI DI COPERTURA – PARETI LATERALI DELL'AULA**	❑
danno	LESIONI VICINE ALLE TESTE DELLE TRAVI LIGNEE, SCORRIMENTO DELLE STESSE – SCONNESSIONI TRA CORDOLI E MURATURA – MOVIMENTI SIGNIFICATIVI DEL MANTO DI COPERTURA	❑❑❑❑❑
20	**MECCANISMI NEGLI ELEMENTI DI COPERTURA – TRANSETTO**	❑
danno	LESIONI VICINE ALLE TESTE DELLE TRAVI LIGNEE, SCORRIMENTO DELLE STESSE – SCONNESSIONI TRA I CORDOLI E MURATURA – MOVIMENTI SIGNIFICATIVI DEL MANTO DI COPERTURA	❑❑❑❑❑
21	**MECCANISMI NEGLI ELEMENTI DI COPERTURA – ABSIDE E PRESBITERIO**	❑
danno	LESIONI VICINE ALLE TESTE DELLE TRAVI LIGNEE, SCORRIMENTO DELLE STESSE – SCONNESSIONI TRA I CORDOLI E MURATURA – MOVIMENTI SIGNIFICATIVI DEL MANTO DI COPERTURA	❑❑❑❑❑
22	**RIBALTAMENTO DELLE CAPPELLE**	❑
danno	DISTACCO DELLA PARETE FRONTALE DALLE PARETI LATERALI	❑❑❑❑❑
23	**MECCANISMI DI TAGLIO NELLE PARETI DELLE CAPPELLE**	❑
danno	LESIONI INCLINATE (SINGOLE O INCROCIATE) – LESIONI IN CORRISPONDENZA DI DISCONTINUITÀ MURARIE	❑❑❑❑❑
24	**VOLTE DELLE CAPPELLE**	❑
danno	LESIONI NELLE VOLTE O SCONNESSIONI DALLE PARETI LATERALI	❑❑❑❑❑
25	**INTERAZIONI IN PROSSIMITÀ DI IRREGOLARITÀ PLANO-ALTIMETRICHE (CORPI ADIACENTI, ARCHI RAMPANTI)**	❑
danno	MOVIMENTO IN CORRISPONDENZA DI DISCONTINUITÀ COSTRUTTIVE - LESIONI NELLA MURATURA PER MARTELLAMENTO	❑❑❑❑❑
26	**AGGETTI (VELA, GUGLIE, PINNACOLI, STATUE)**	❑
danno	EVIDENZA DI ROTAZIONI PERMANENTI O SCORRIMENTO – LESIONI	❑❑❑❑❑
27	**TORRE CAMPANARIA**	❑
danno	LESIONI VICINO ALLO STACCO DAL CORPO DELLA CHIESA – LESIONI A TAGLIO O SCORRIMENTO – LESIONI VERTICALI O ARCUATE (ESPULSIONE DI UNO O PIÙ ANGOLI)	❑❑❑❑❑
28	**CELLA CAMPANARIA**	❑
danno	LESIONI NEGLI ARCHI – ROTAZIONI O SCORRIMENTI DEI PIEDRITTI	❑❑❑❑❑

A_{17} - INDICE DI DANNO

n = ⌊__⌋__ (numero dei meccanismi possibili) d = ⌊__⌋__ (punteggio totale di danno) $i_d = d / 5n$ = ⌊__⌋__

A_{18} - AGIBILITA'

Agibile ❍	Inagibile ❍

Parzialmente Agibile ❍	Agibile con Provvedimenti ❍	Temporaneamente Inagibile ❍	Inagibile per cause Esterne ❍
Indicare le parti agibili	Segnalare i provvedimenti anche indicandoli nella tabella sottostante	❑ Verifica più accurata ❑ Si consiglia visita di esperti ❑ Altro	Indicare le cause esterne
....	....	....	....
....	....	....	....
....	....	....	....
....	....	....	....
....	....	....	....
....	....	....	....
....	....	....	....

A_{19} – TIPO DI VISITA

Completa ❍	Parziale ❍	Solo dall'esterno ❍	Motivi ostativi ....

A_{20} - PROVVEDIMENTI DI P.I. SUGGERITI (* interventi limitati ** interventi estesi)

	PROVVEDIMENTI	*	**		PROVVEDIMENTI	*	**
1	Revisione manto di copertura	❑	❑	8	Ripristino smaltimento delle acque meteoriche	❑	❑
2	Copertura provvisoria	❑	❑	9	Monitoraggio	❑	❑
3	Puntellamenti	❑	❑	10	Protezioni o consolidamenti su opere d'arte fisse	❑	❑
4	Rimozione delle macerie	❑	❑	11	Catalogazione e smontaggio delle parti pericolanti	❑	❑
5	Transennamenti / recinzioni / protezioni	❑	❑	12	Sgombero opere d'arte mobili	❑	❑
6	Consolidamenti localizzati	❑	❑	13	Raccolta sistematica dei frammenti	❑	❑
7	Messa in opera di cerchiatura e/o tiranti	❑	❑	14	Ricovero e protezione dei frammenti	❑	❑

A_{21} - DANNI ALL'APPARATO DECORATIVO E ALLE OPERE D'ARTE *(scheda dettagliata a parte)*

$A_{21.1}$ - Descrizione apparato decorativo o opera d'arte

....

....

....

$A_{21.2}$ - Descrizione danno

....

....

....

$A_{21.3}$ - Provvedimenti provvisionali sugli apparati decorativi e sgombero opere d'arte mobili

SI CONSIGLIA INTERVENTO STORICO DELL'ARTE ❑

....

....

....

STIMA DEL COSTO PER LA SALVAGUARDIA DELLE OPERE D'ARTE

€ __|__|__| __|__|__| **,00**

A$_{22}$ - DESCRIZIONE E STIMA SOMMARIA DELLE OPERE NECESSARIE

A$_{22.1}$ - Descrizione opere di ripristino strutturale (nuovi danni e danni pregressi aggravati)

....

....

....

STIMA DEL COSTO PER IL RIPRISTINO STRUTTURALE

€ |_|_|_| |_|_|_| |_|_|_| ,00

A$_{22.2}$ - Descrizione opere di finitura, impiantistica e miglioramento sismico collegate

....

....

....

STIMA DEL COSTO OPERE FINITURA IMPIANTISTICA E MIGLIORAMENTO SISMICO

€ |_|_|_| |_|_|_| |_|_|_| ,00

A$_{22.3}$ - Descrizione opere di pronto intervento (eventualmente indicare anche il costo del P.I. **"a finire"**)

....

....

....

STIMA DEL COSTO OPERE DI PRONTO INTERVENTO

€ |_|_|_| |_|_|_| ,00

A$_{23}$ - NOTE

Indicare, eventualmente, altri danni non rilevabili dalla scheda (es. solai di calpestio, pavimentazioni ecc.)

....

....

....

....

....

....

....

....

....

A$_{24}$ - DATI DIMENSIONALI (stimati ❍ rilevati ❍)

Aula (compresi navate, cappelle, transetti)	Larghezza mt. |_|_|_|	Lunghezza mt. |_|_|_|	Superficie mq. |_|_|_|_|	Altezza media mt. |_|_|_|
Abside	Larghezza mt. |_|_|_|	Lunghezza mt. |_|_|_|	Superficie mq. |_|_|_|_|	Altezza media mt. |_|_|_|
Facciata principale	Larghezza mt. |_|_|_|	Altezza mt. |_|_|_|	Superficie mq. |_|_|_|_|	
Campanile	Larghezza mt. |_|_|_|	Lunghezza mt. |_|_|_|		Altezza mt. |_|_|_|
Coperture chiesa	Larghezza mt. |_|_|_|	Lunghezza mt. |_|_|_|	Superficie mq. |_|_|_|_|	Altezza massima mt. |_|_|_|

A$_{25}$ - ELABORATI GRAFICI (piante, sezioni, prospetti, illustrazione di dissesti particolari, allegare eventualmente fotocopie)

A$_{26}$ – DOCUMENTAZIONE ALLEGATA SI ❍ NO ❍

....

....

....

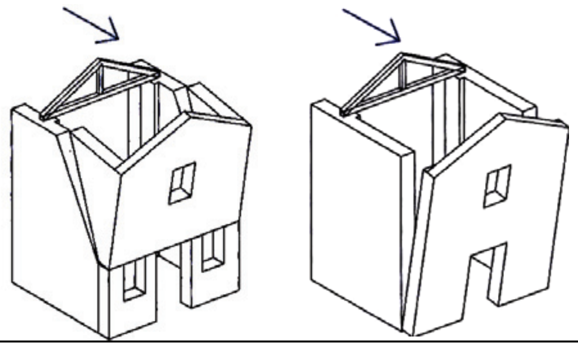
A$_{27}$ - SQUADRA CHE HA ESEGUITO IL RILIEVO

SISMA		C.O.M.	SQUADRA N.
Componenti della squadra			
Cognome e nome	Qualifica	Ente appartenenza	Firma

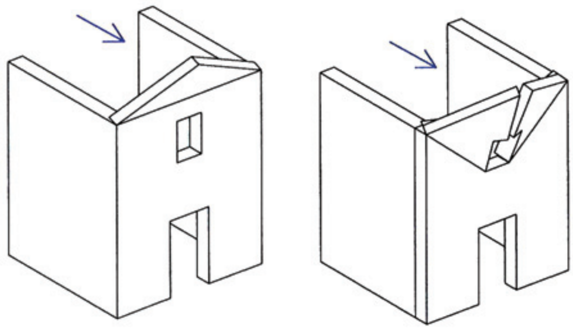
Modello A-DC PCM-DPC MiBAC 2006

ABACO DEI MECCANISMI DI COLLASSO DELLE CHIESE

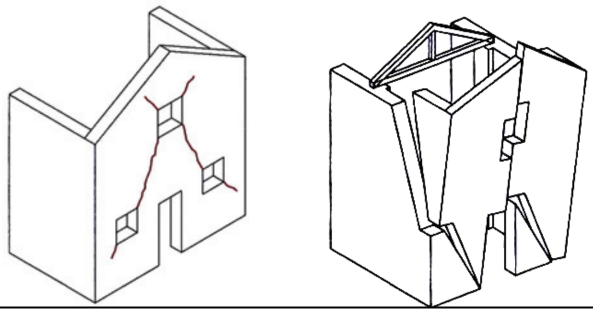
1. RIBALTAMENTO DELLA FACCIATA



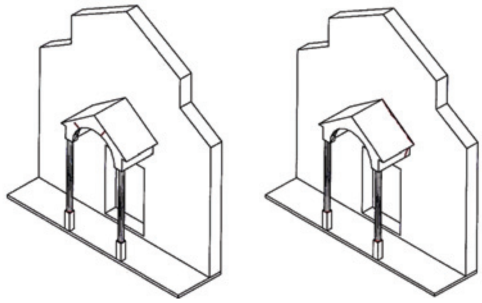
2. MECCANISMI NELLA SOMMITÀ DELLA FACCIATA



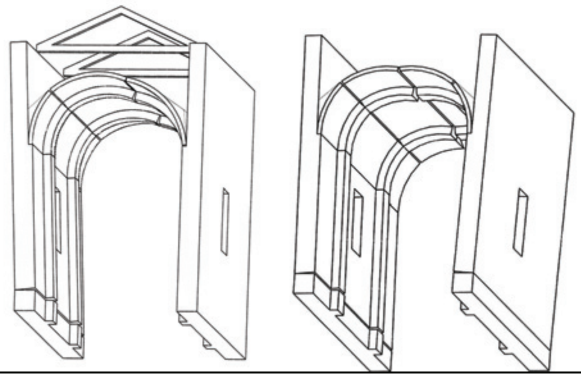
3. MECCANISMI NEL PIANO DELLA FACCIATA



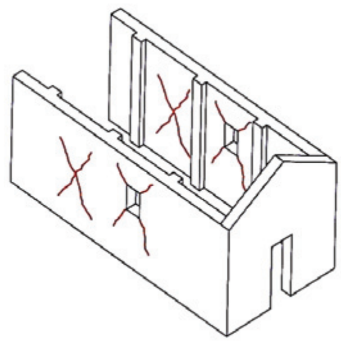
4 - PROTIRO E NARTECE



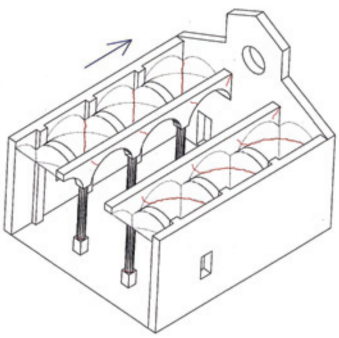
5 - RISPOSTA TRASVERSALE DELL'AULA



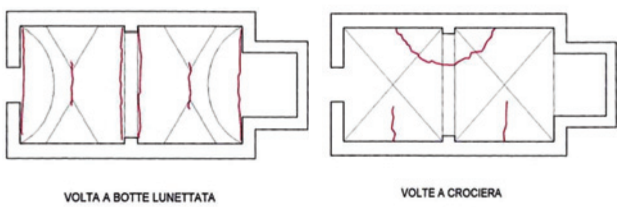
6 - MECCANISMI DI TAGLIO PARETI LATERALI



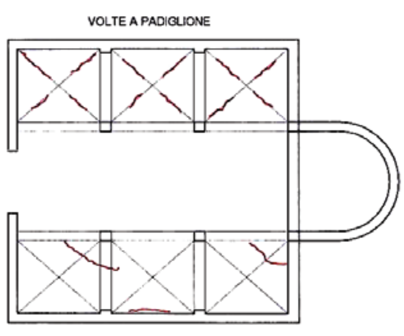
7 - RISPOSTA LONGITUDINALE DEL COLONNATO



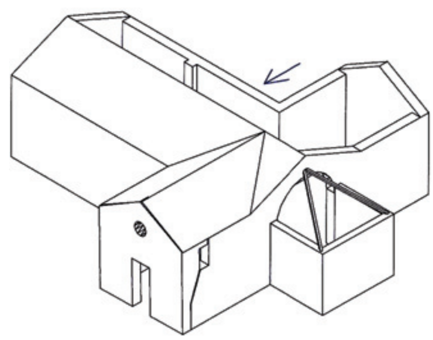
8 - VOLTE DELL'AULA O DELLA NAVATA CENTRALE



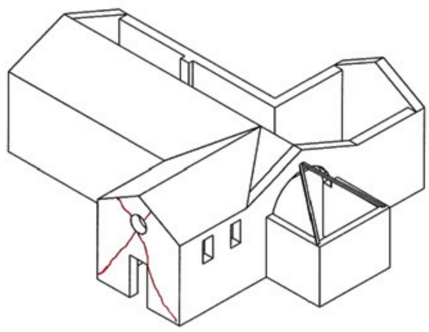
9 - VOLTE DELLE NAVATE LATERALI



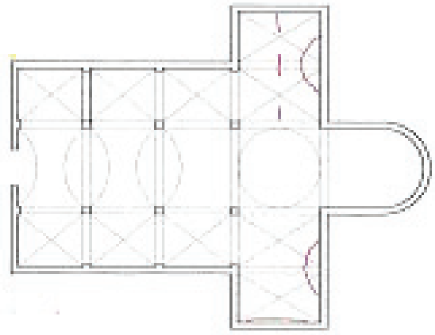
10 - RIBALTAMENTO PARETI DEL TRANSETTO



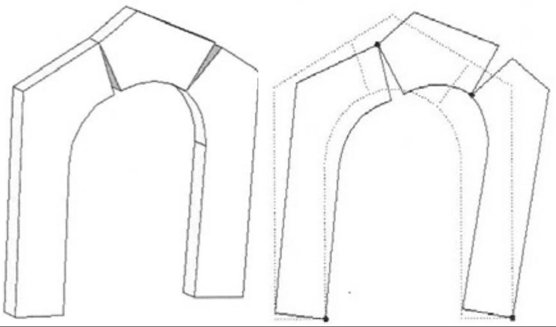
11 - MECCANISMI DI TAGLIO DEL TRANSETTO



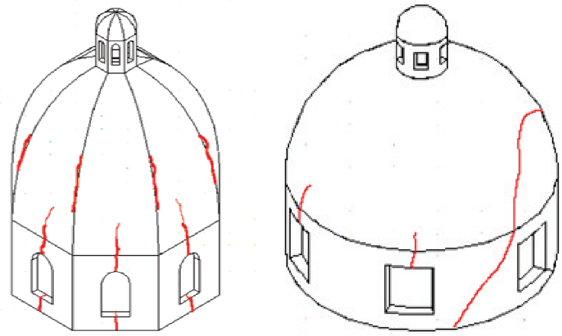
12 - VOLTE DEL TRANSETTO



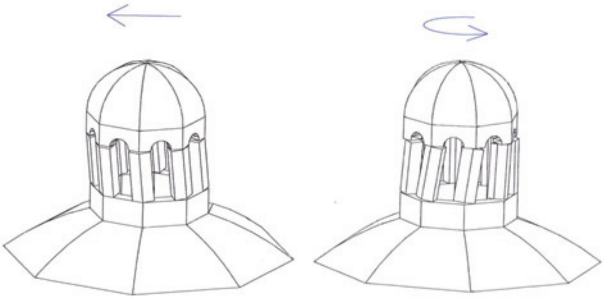
13 - ARCHI TRIONFALI



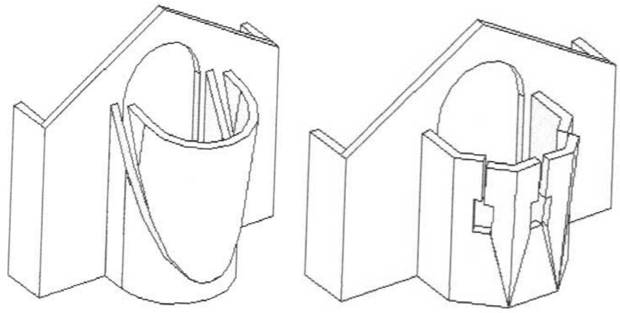
14 - CUPOLA E TAMBURO / TIBURIO



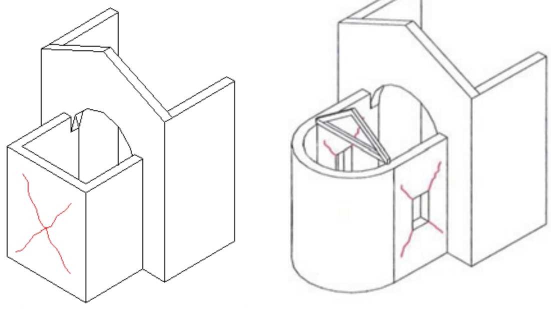
15 – LANTERNA



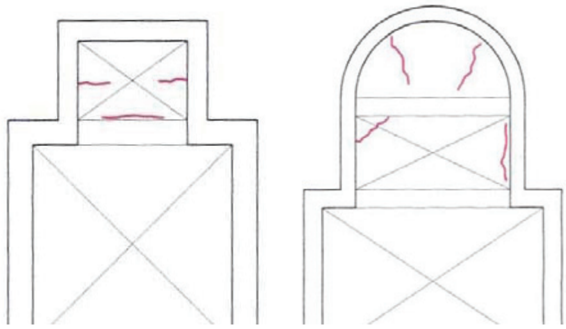
16 - RIBALTAMENTO DELL'ABSIDE



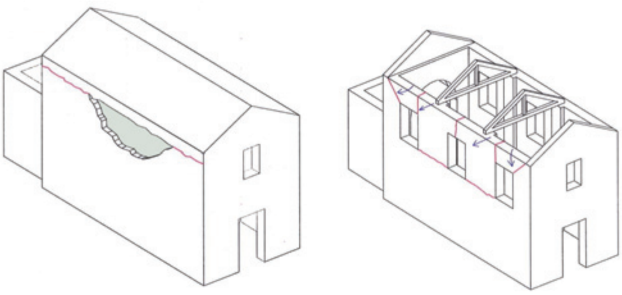
17 - MECCANISMI DI TAGLIO NELL'ABSIDE



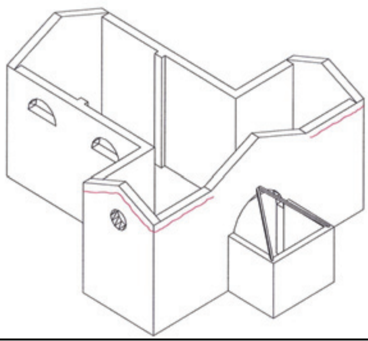
18 - VOLTE DEL PRESBITERIO O DELL'ABSIDE

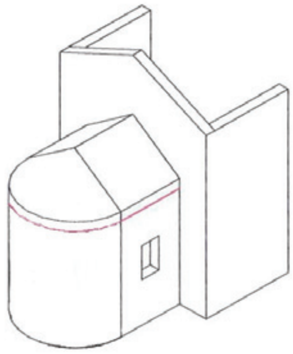
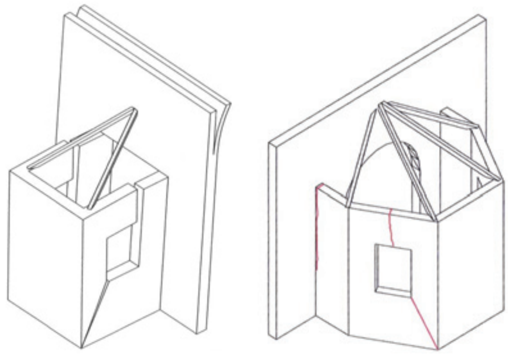
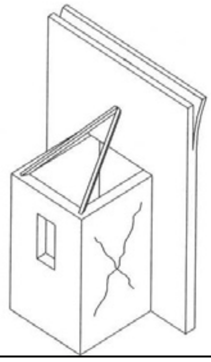
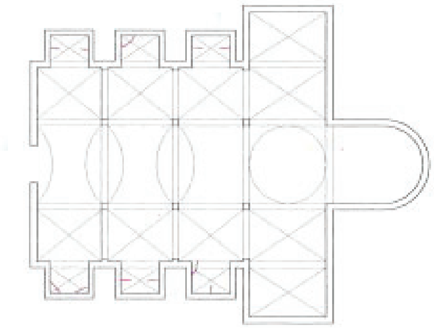
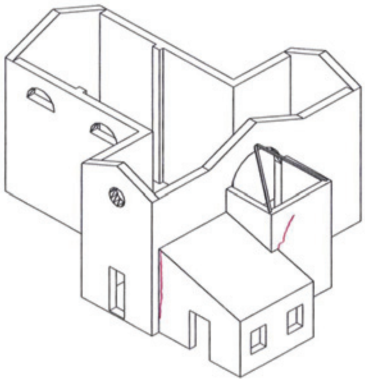
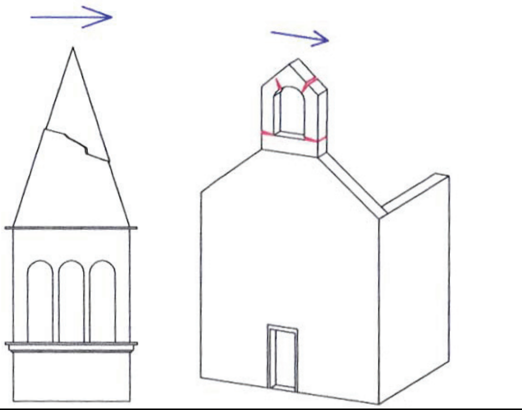
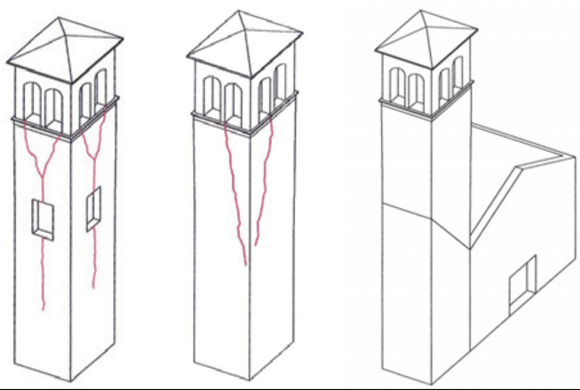
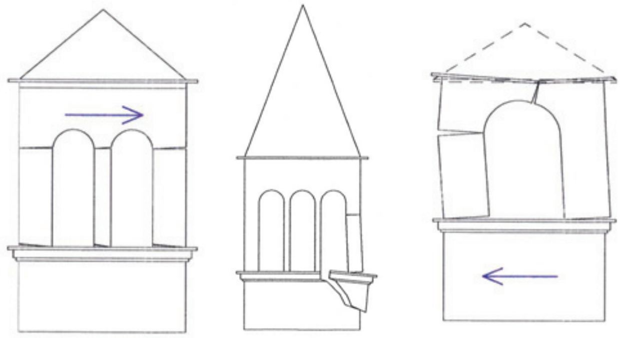


19 – ELEMENTI DI COPERTURA: AULA



20 - ELEMENTI DI COPERTURA: TRANSETTO



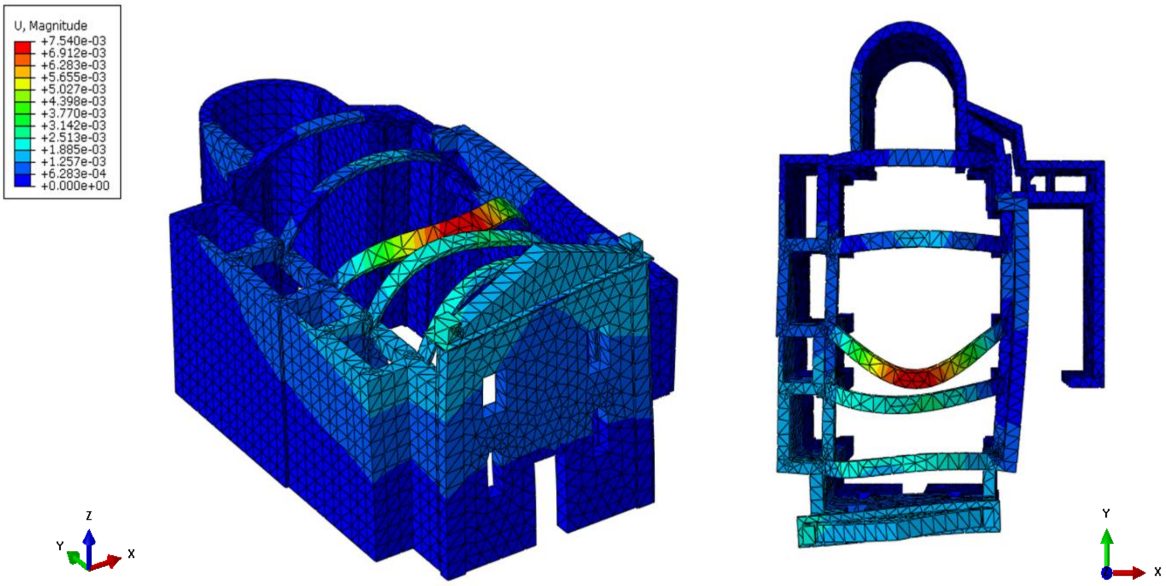
21 - ELEMENTI DI COPERTURA: ABSIDE	**22 - RIBALTAMENTO DELLE CAPPELLE**
	
23 - MECCANISMI DI TAGLIO NELLE CAPPELLE	**24 - VOLTE DELLE CAPPELLE**
	
25 - INTERAZIONI IN PROSSIMITA' DI IRREGOLARITÀ	**26 - AGGETTI (VELA, GUGLIE, PINNACOLI, STATUE)**
	
27 - TORRE CAMPANARIA	**28 - CELLA CAMPANARIA**
	

Modello A-DC PCM-DPC MiBAC 2006

APPENDIX A

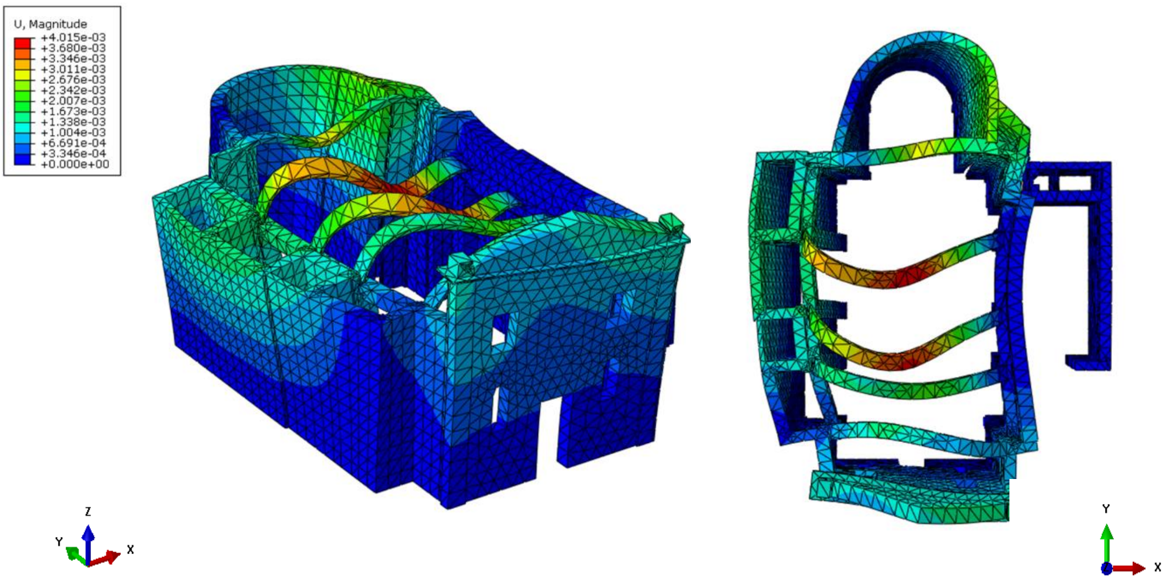
Mode 7

T [s]: 0.16



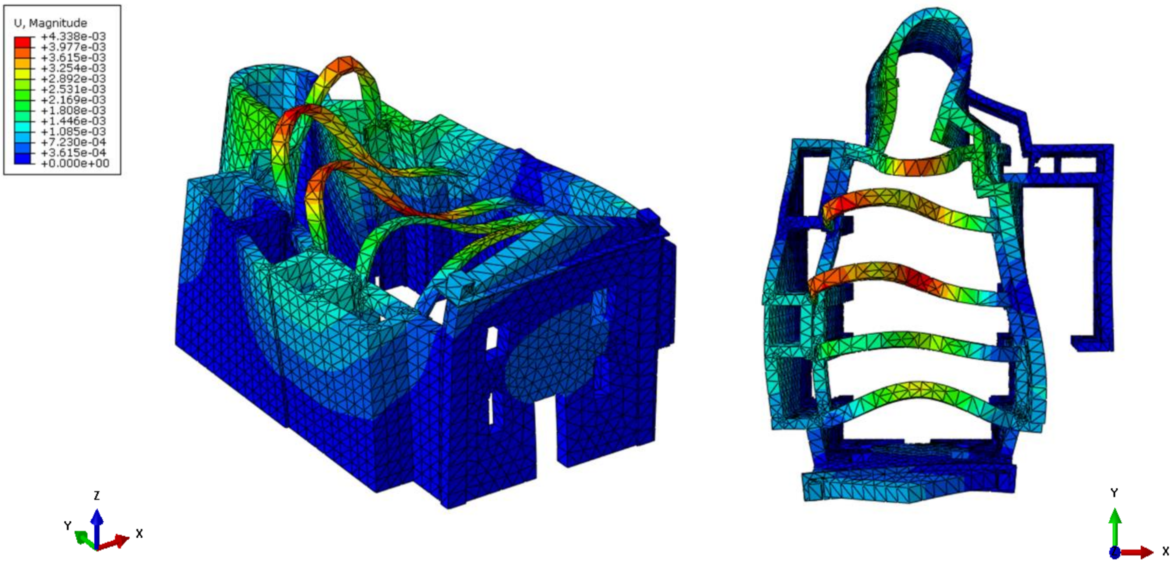
Mode 8

T [s]: 0.15



Mode 9

T [s]: 0.14

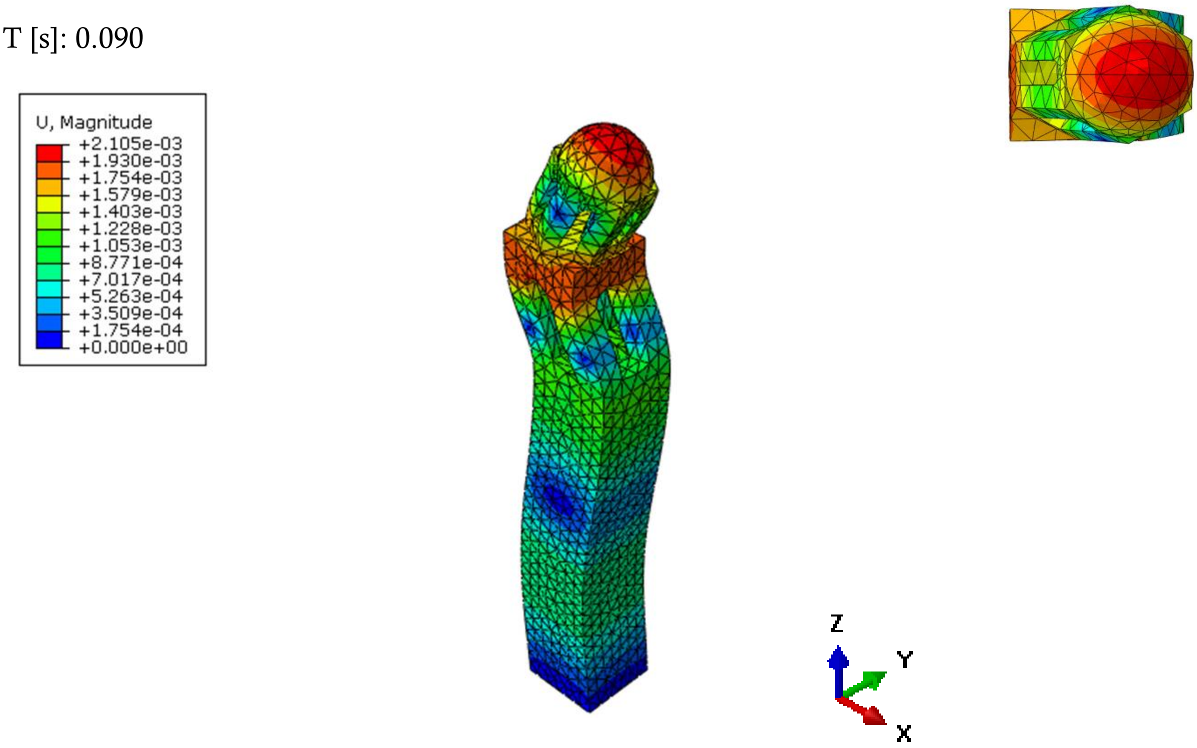

U, Magnitude
+4.338e-03
+3.977e-03
+3.615e-03
+3.254e-03
+2.892e-03
+2.531e-03
+2.169e-03
+1.808e-03
+1.446e-03
+1.085e-03
+7.230e-04
+3.615e-04
+0.000e+00
Z
Y
X
Y
X

APPENDIX B

Mode 11

F [Hz]: 11.023

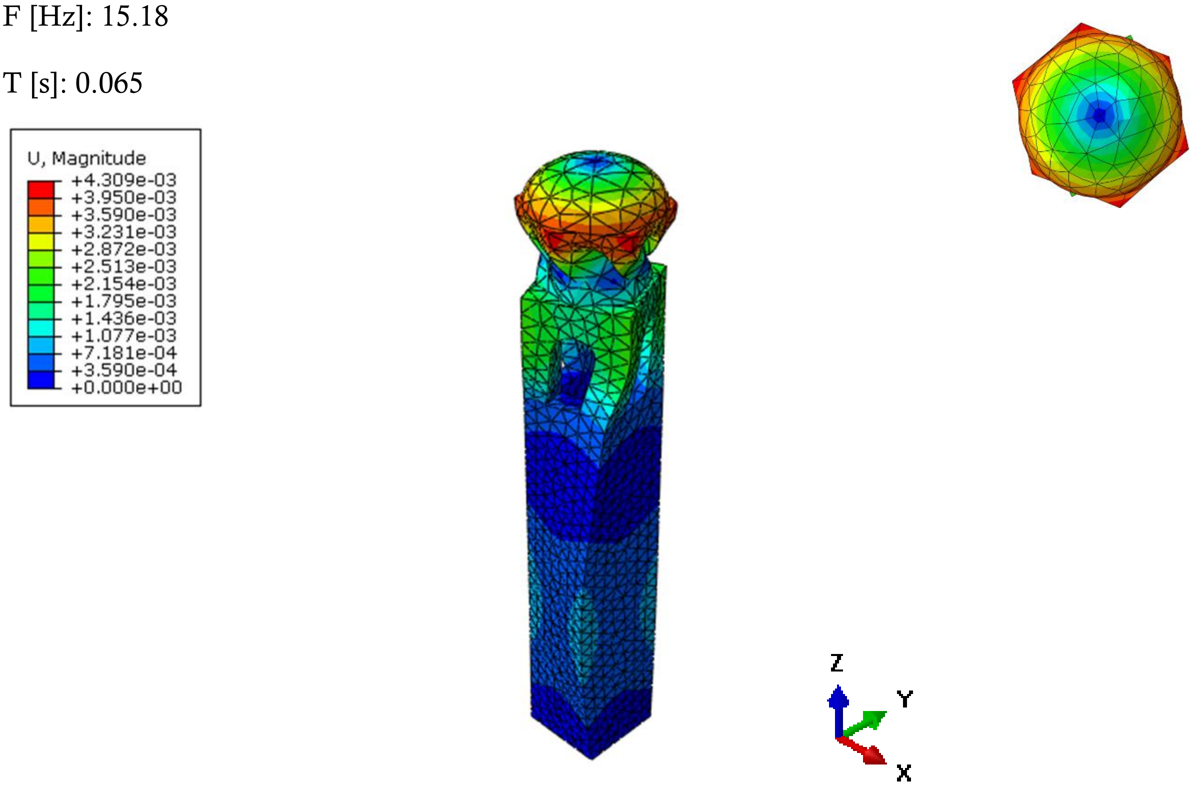
T [s]: 0.090



Mode 12

F [Hz]: 15.18

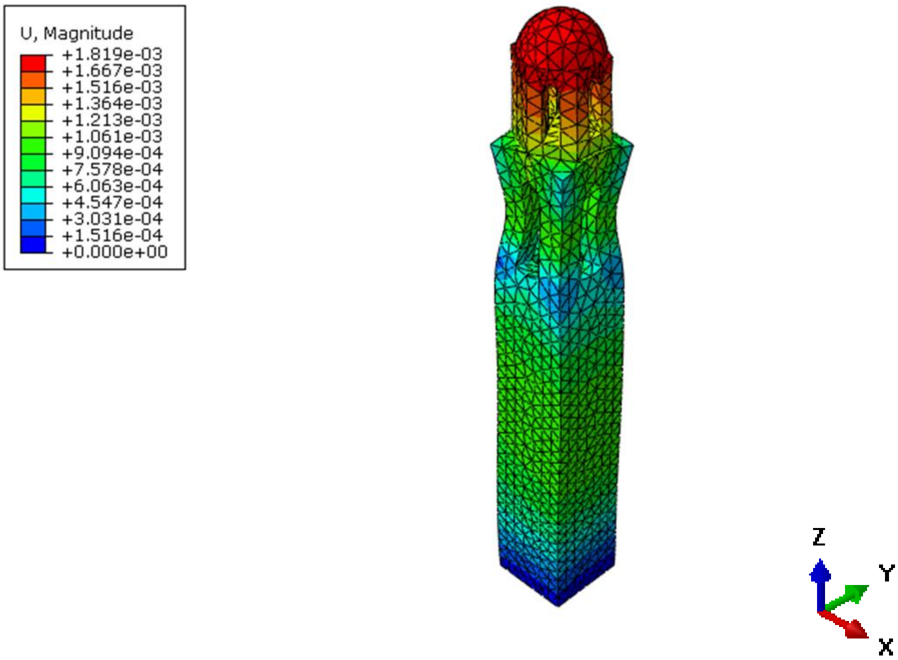
T [s]: 0.065

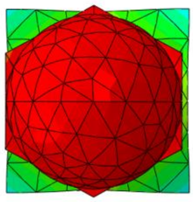


Mode 13

F [Hz]: 17.388

T [s]: 0.057

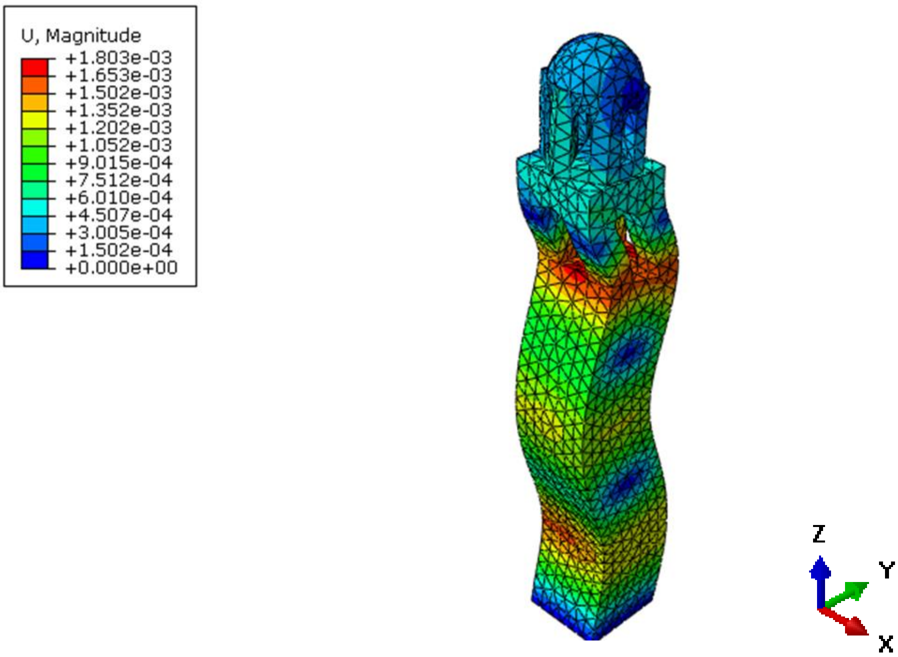


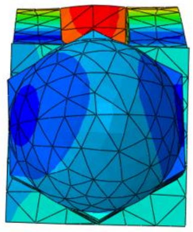


Mode 14

F [Hz]: 17.787

T [s]: 0.056

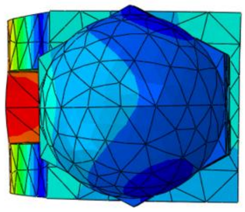


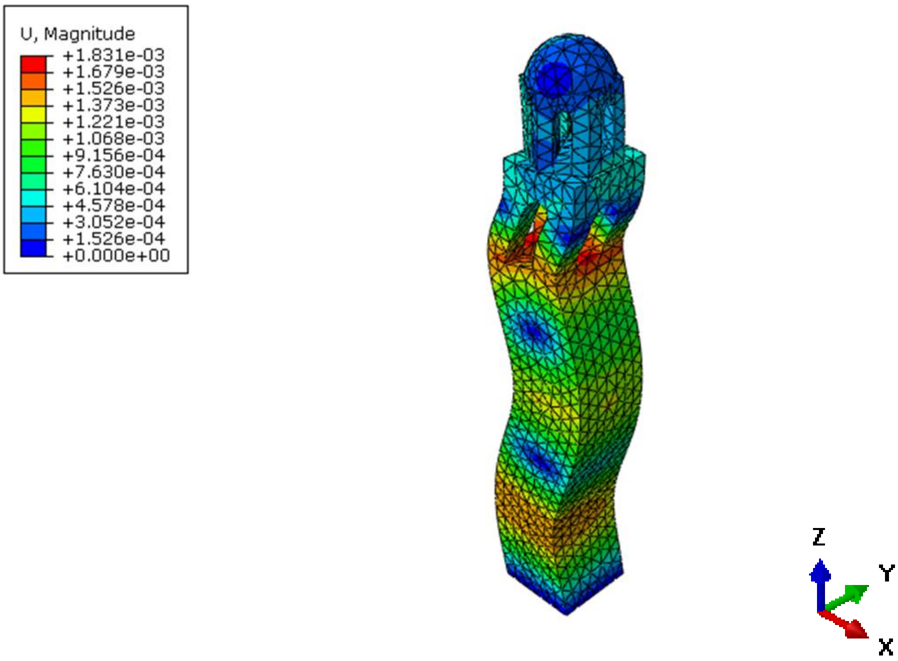


Mode 15

F [Hz]: 17.942

T [s]: 0.055

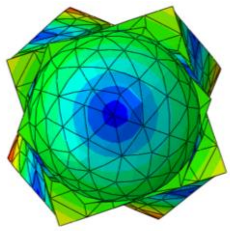


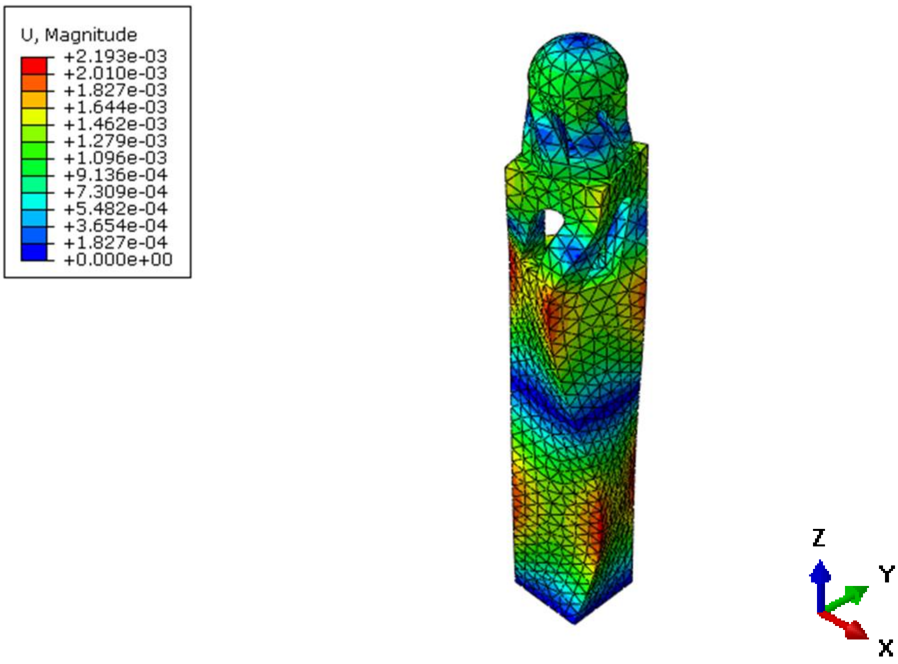


Mode 16

F [Hz]: 18.427

T [s]: 0.054

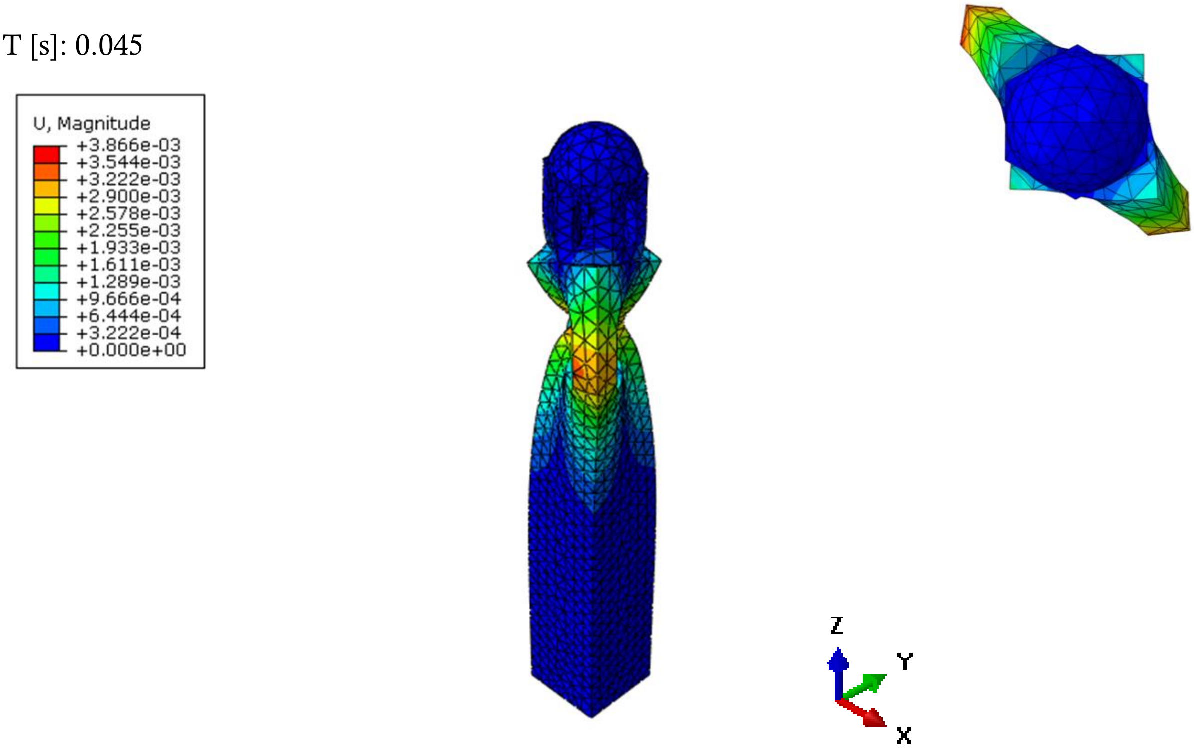




Mode 17

F [Hz]: 21.761

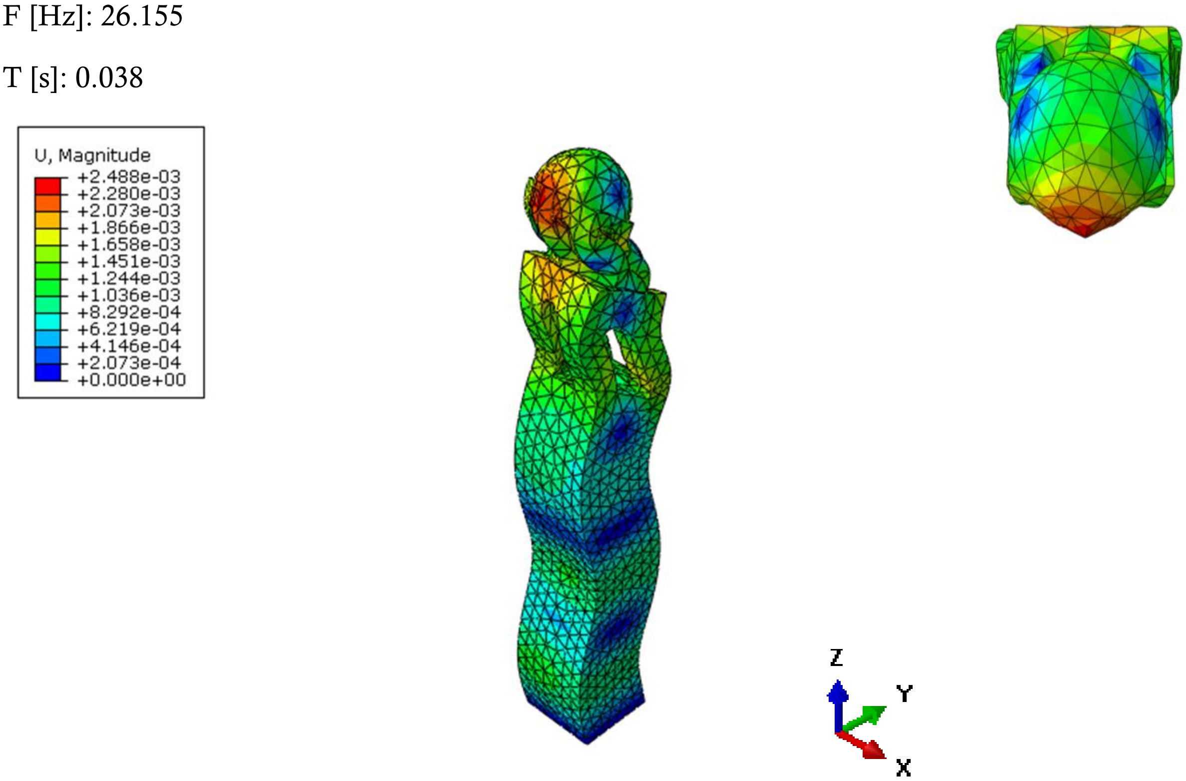
T [s]: 0.045



Mode 18

F [Hz]: 26.155

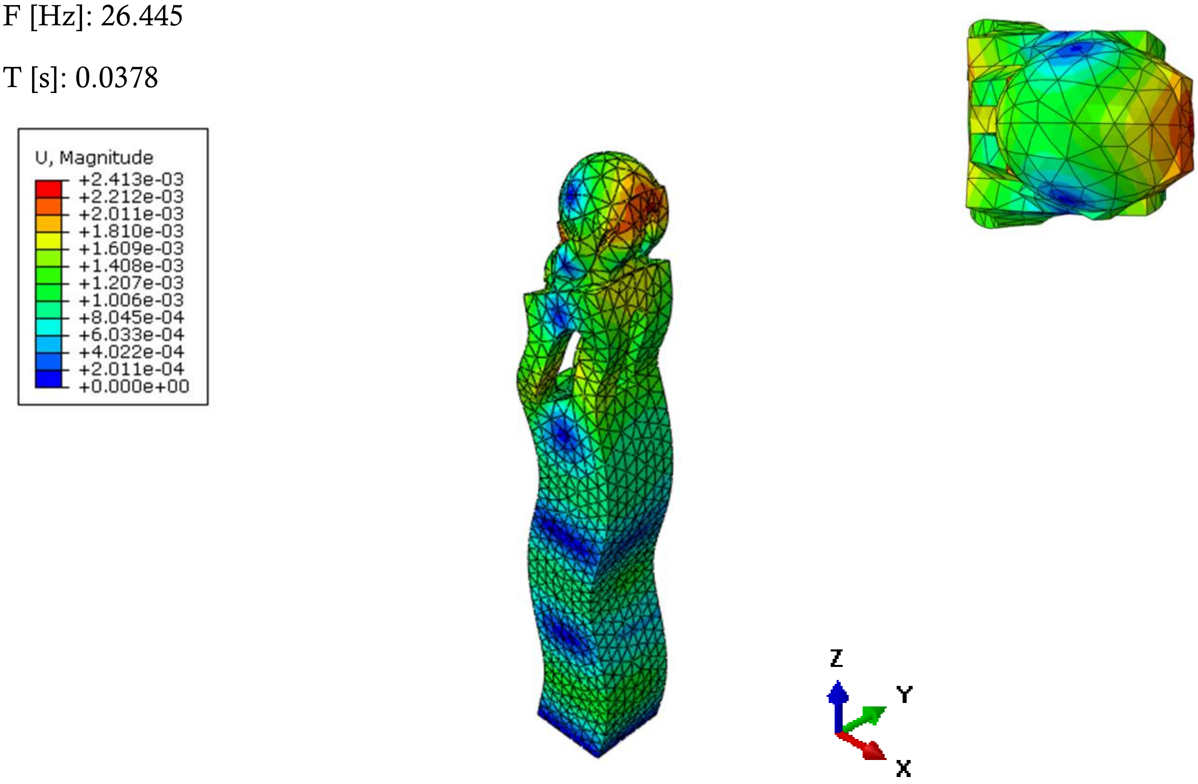
T [s]: 0.038



Mode 19

F [Hz]: 26.445

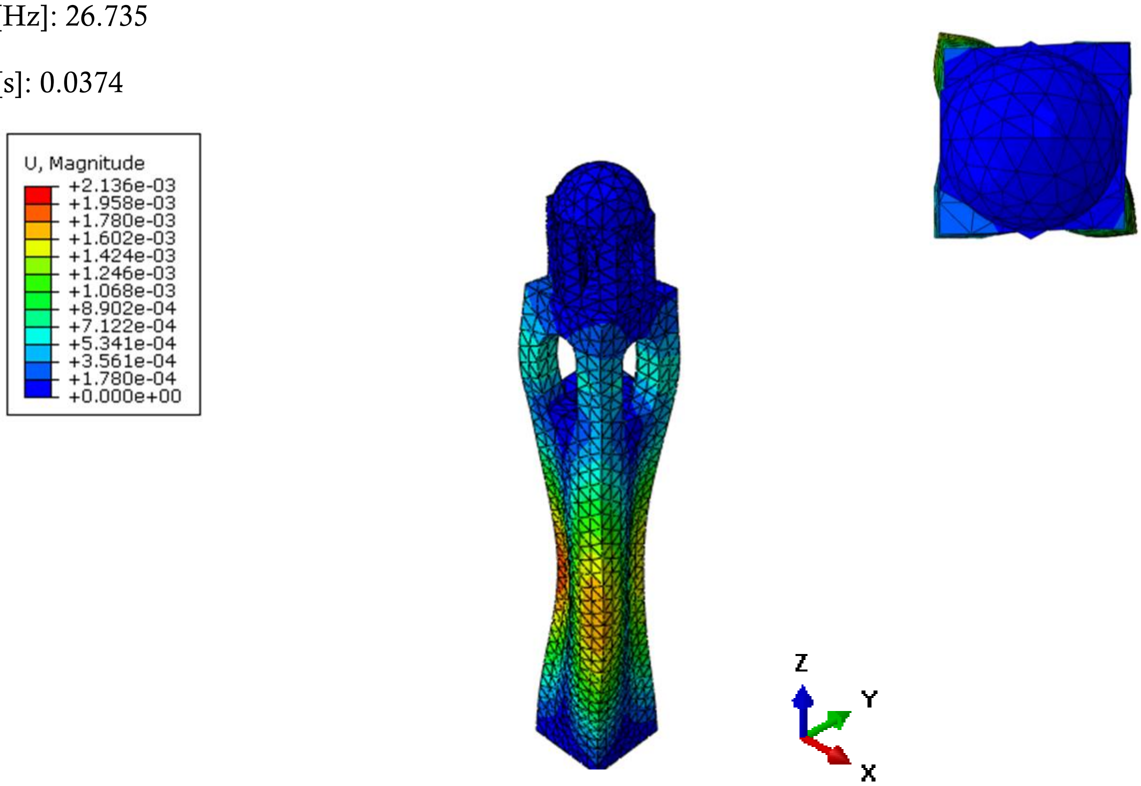
T [s]: 0.0378



Mode 20

F [Hz]: 26.735

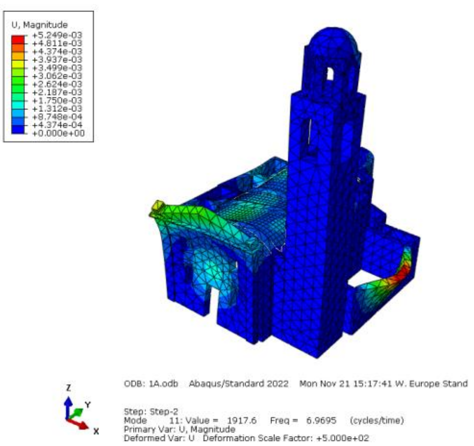
T [s]: 0.0374

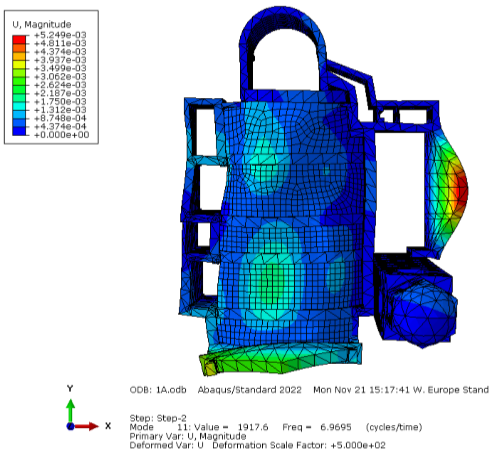


APPENDIX C

Mode 11

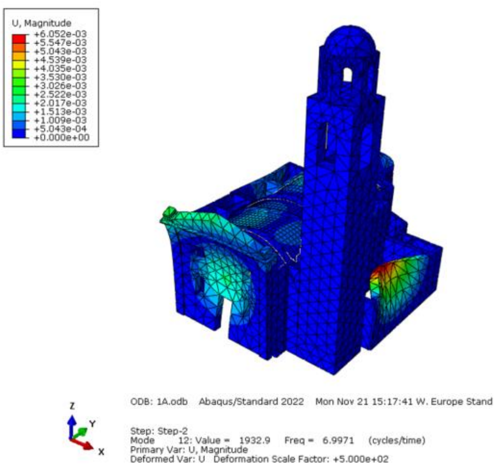
T [s]: 0.1435

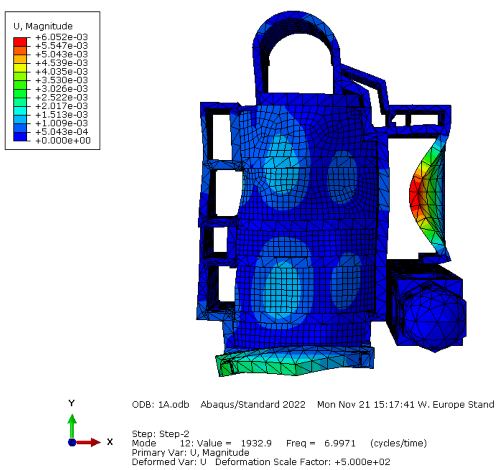




Mode 12

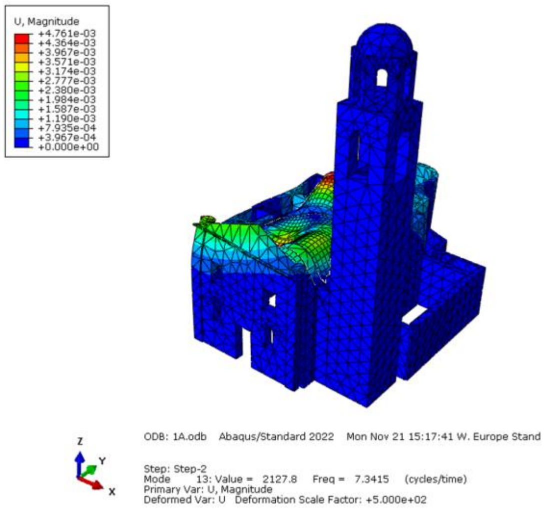
T [s]: 0.1432

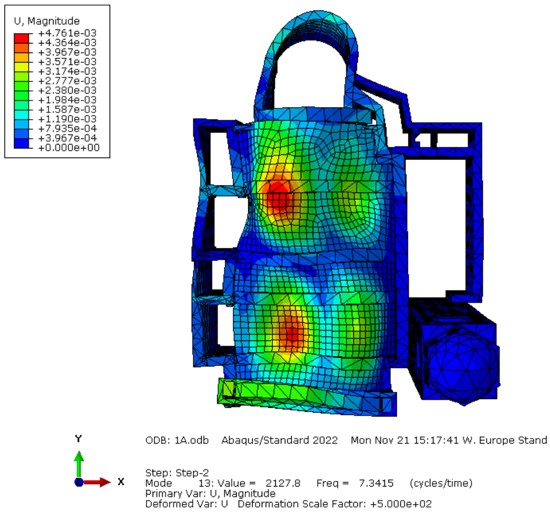




Mode 13

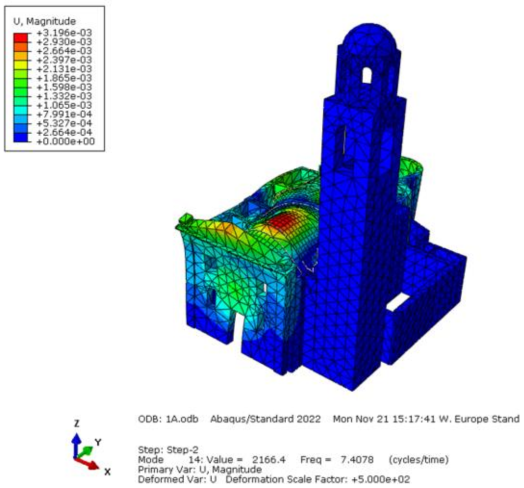
T [s]: 0.136

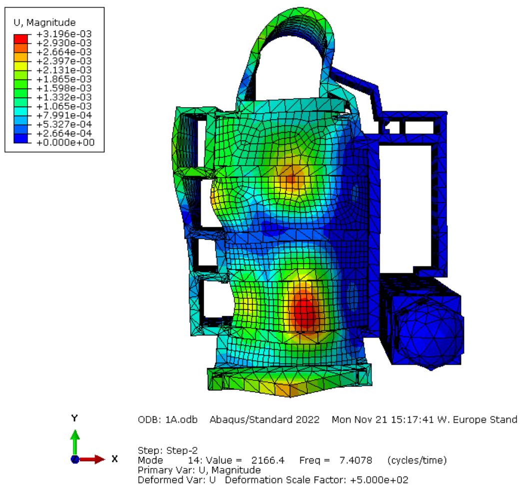




Mode 14

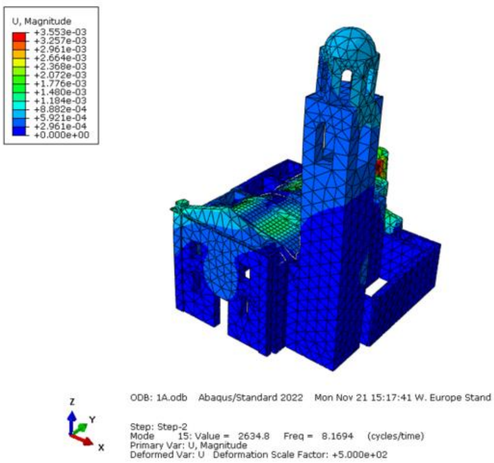
T [s]: 0.13499

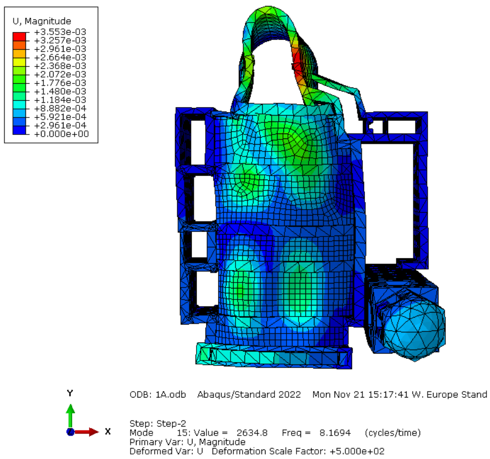




Mode 15

T [s]: 0.122





Mode 16

T [s]: 0.1216