



## MASTER'S THESIS PROPOSAL

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Department: Department of Mechanics  
Thesis supervisor: Prof. Ing. Pavel Kuklik  
Thesis title: Nonlinear numerical evaluation of the bearing capacity and the structure stability of the St. Jacob Church from the Broumov Group of Churches  
Thesis title in English: see above

### Framework content:

The main objective of the thesis is the FEM modelling and the subsequent evaluation of bearing capacity of St. Jacob Church in the Broumov Region, with a particular attention on the influence of the shallow footing and their degradation.

The thesis, after an historical introduction, focused on the several damages experienced by the church, trying to define the possible causes.

After this, as the presence of a peculiar arrangement of stones, a micromodelling of different portion of wall was carried out by means of ATENA, in order to obtain mechanical parameters to be used in the 3D model.

The 3D model in DIANA explored different scenarios, studying the influence on the structure of possible future differential settlement. Moreover, an explanation of the currently cracking situation is provided.

In conclusion, suggestions in order to decrease the number of uncertainties are provided, along with monitoring strategies.

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## DECLARATION

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Title of the Msc Dissertation: Nonlinear numerical evaluation of the bearing capacity and the structure stability of the St. Jacob Church from the Broumov Group of Churches

Supervisor(s): Prof. Ing. Pavel Kuklik

Year: 2017/2018

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To my SAHC friends.

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## ABSTRACT

Broumov Region and its group of Baroque churches represents a perfect example of combination between architecture and rural landscape. This peculiar heritage site requires specialized analysis in order to preserve its uniqueness. St. Jacob Church, the one investigated in this thesis, was the first one to be built in the region by Crisptoph and Kilian Dientzenhofer.

Nowadays, concerns about its stability and degradation process led to the necessity to investigate more deeply its current situation, affected by several uncertain. So, the main objective of the thesis consisted in the FEM modelling and the subsequent evaluation of bearing capacity of enclosure walls, with a particular attention on the influence of the soil degradation.

The thesis, after an historical introduction, focused on the several damages experienced by the church, trying to define the possible causes. A detailed register of the damages was finally realized.

After this, as the presence of a peculiar arrangement of stones, a micromodelling of different portion of wall was carried out by means of ATENA 2D. This approach allowed to obtain mechanical parameters, with values comparable to reliable standards, to be used afterwards for the 3D model in DIANA.

The information available for the soil properties allowed to determine the overall settlement of the structure, through simple elastic analysis with GEO5 and DIANA. The values obtained with these two different approaches resulted to be similar.

After this, the non linear analysis explored different scenarios, studying the influence on the structure of possible future differential settlements, creating a catalogue useful for future analysis. Moreover, a reasonable explanation of the currently cracking pattern is provided.

In conclusion, technical advice in order to decrease the number of uncertain are suggested, along with monitoring strategies.

**Keywords:** Baroque, soil structure interaction, bearing capacity, crack propagation, cultural heritage, stability.

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## ABSTRAKT

### **Nelineární analýzy vyhodnocení únosnosti a stabilita z Kostel svatého Jakuba v Broumovsko.**

Broumovsko a jeho skupina barokních kostelů představuje výjimečný příklad kombinace architektury a venkovské krajiny. Toto zvláštní dědictví vyžaduje speciální odbornou analýzu, aby byla zachována jeho jedinečnost. Kostel svatého Jakuba, který byl zkoumán v této diplomové práci, byl prvním, který v regionu vybudovali Kryštof a Kilián Dientzenhoferové.

Dnešní obavy, týkající se jeho stability a stavu degradace, vedly k nutnosti hlouběji prozkoumat jeho současnou situaci, která je ovlivněna řadou nejistot. Hlavním cílem práce byla tedy modelování MKP a následné vyhodnocení únosnosti stěn obvodového pláště se zaměřením na vliv degradace podloží.

Tato práce, po historickém úvodu, se zaměřila na vybrané poruchy, které jsou na kostele patrné, a stanovení jejich příčin. Poškození bylo podrobně zaznamenáno.

Následně, zvoleným zvláštním uspořádáním kamenných částí zdiva, bylo pomocí software ATENA 2D provedeno určité mikromodelování různých částí zdi. Toto řešení umožnilo získat mechanické parametry, jež jsou srovnatelné se spolehlivými normami. Tyto hodnoty byly následně použity pro 3D modelování užitím numerického kódu DIANA.

Dostupné informace o vlastnostech podloží nám umožnily stanovit celkové sedání stavby pomocí elastické analýzy pomocí kódů GEO5 a DIANA. Získané hodnoty sedání pomocí obou kódů vykazovaly minimální odchylky.

Poté pomocí nelineární analýzy jsme prozkoumali různé scénáře zaměřené na nerovnoměrné sedání. Byl vytvořen určitý katalog možných poruch, který bude užitečný pro další analýzu. Mimo to se nám podařilo uspokojivě vysvětlit současný stav porušení prezentovaný sadou zjevných trhlin.

Na závěrem je navrženo technické opatření zaměřené na redukování nejistot současného stavu spolu se strategií sledování kostela.

**Klíčová slova:** Baroko, interakce stavby s podložím, únosnost, šíření trhliny, kulturní dědictví, stabilita.

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## SOMMARIO

### **Calcolo non lineare della capacità portante e della stabilità della Chiesa di San Jacopo nella regione del Broumov.**

La regione del Broumov e il suo insieme di chiese barocche rappresentano un grande esempio di perfetta integrazione tra architettura e paesaggio. La particolarità del posto richiede studi avanzati in modo da preservare la sua unicità. La chiesa di San Jacopo, trattata in questo lavoro di tesi, è stata la prima ad essere realizzata nella regione da Crisptoph and Kilian Dientzenhofer.

Le preoccupazioni riguardo la stabilità e il decadimento della chiesa hanno comportato la necessità di investigare più in dettaglio l'attuale situazione, condizionata da diverse incertezze. Il principale obiettivo della tesi è stata la realizzazione di un modello FEM e la analisi della capacità portante delle pareti della chiesa, tenendo conto dell'influenza del deterioramento del suolo.

La tesi, dopo una introduzione storica, si è concentrata sui numerosi danni riscontrati nella chiesa, provando a definirne le possibili cause. Un dettagliato schedario dei danni è stato infine compilato.

In seguito, per via della particolare tipologia dei muri, sono state realizzate tramite ATENA 2D analisi di micromodellazione di differenti porzioni delle pareti esterne. Ciò ha permesso di ottenere parametri meccanici della muratura, con valori vicini ad attendibili norme, da poter usare successivamente nel modello 3D in DIANA.

Le informazioni disponibili riguardanti le proprietà del suolo hanno permesso di calcolare il globale spostamento verticale della struttura, tramite semplici analisi elastiche con GEO5 e DIANA. I valori ottenuti, con i due diversi programmi, sono risultati molto vicini.

Successivamente analisi non lineari sono state eseguite in modo da esplorare differenti scenari, studiando l'influenza sulla struttura di possibili futuri cedimenti differenziali. Inoltre, sono state elaborate ragionevoli spiegazioni sulle cause delle attuali fessure.

Per concludere, sono stati forniti suggerimenti tecnici, con l'obiettivo di ridurre il numero di incertezze, e strategie di monitoraggio.

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## 1. INTRODUCTION

The architectural treasures that make precious the landscape of Broumov region, represent an heritage site to preserve and protect for its uniqueness in the history, in the features and in the concept.

The nine baroque churches, known as Broumov's Churches, were built in the first period of 18<sup>th</sup> century in a surprising short period of 20 years, by the Dientzenhofers, already authors of architectonic masterpieces in Prague. The combination of an almost uncontaminated landscape with the Baroque style led to a marvellous scenario that deserves efforts in order to detect and minimize the currently vulnerabilities of these building.

In this context, the St. Jacob Church in Ruprechtice stands for an important example of the Broumov's churches and it will be the case study investigated in the present thesis.

The structural analysis of St. Jacob is made difficult by the several uncertain that affect the level of knowledge. The doubts regard the materials and the technique used for rising up the enclosure walls, and even the past and currently condition of the soil, that can be seen as a real source of concern.

One of the goals of the thesis is at first to give a contribute in reducing these kind of uncertain or at least to define where is more convenient to carry out further investigations.

The capacity of the enclosure walls will be investigated with several approaches in a non-linear regime, by means of micro modelling, , in order to get useful parameters to be used in the 3D model.

In the second part of the thesis, the analysis with 3D model will be strictly linked to the issues of poor soil condition with a focus on possible explanations of current cracks.

Furthermore, the goal is also to provide a sort of catalogue of possible future cracks, exploring different scenarios related to soil settlements in order to make easy the identification of possible phenomena.



## **2. HISTORICAL OVERVIEW**

### **2.1. HISTORICAL CONTEXT AND BAROQUE**

The events related to the Protestant Reformation and the consequent Catholic Counter Reformation, occurred in the 16<sup>th</sup> century, had a huge impact on the visual arts and the architecture of 17<sup>th</sup> and 18<sup>th</sup> centuries.

In Europe the National States had established, and the Thirty Years War led to the decline of Spanish Empire with the France achieving more power. Meanwhile, Italy experienced a period of economic recession, and the Papacy was trying to defence and regain his role as reference.

In this peculiar context, starting from Rome, and artistic revolution, the so called Baroque, spread all over the Europe, giving new aspect to several cities (Gattuso, 2013).

The Baroque gained a fundamental role in the process of Counter Reformation. It is important to highlight as the new scientific findings of the previous century enabled to improve the human knowledge. This aspect, along with the considerable criticism by the reformists, led the Church to resort even more to the force of persuasion. The peculiar style of Baroque, inventive, emotive, opulent, looked as the perfect vehicle in order to communicate ideologies and impress the believers.

From the architectonic point of view, the main difference with the past styles, is a new conceptions of the space, that could be shaped without anymore the strict rules of 15<sup>th</sup> century, playing with more complex and dynamic plants. Curve lines are the distinctive aspect, but much more irrational than the ones adopted in the Renaissance, preferring ellipsis, paraboles, hyperboles (Gattuso, 2013).

As said, the goal was to impress, so the conception of the building is designed taking into account the different perspectives. Also the façade, is not any more correlated with the internal distribution of the navies but acquired a new meaning in relationship with the context, becoming an element of urban design. Even if the relationship between Counter Reformation and Baroque could have been empathized too much in the literature, certainly the Jesuits, in their process of proselytism, adopted this style and exported it all over the Europe.

### **2.2. BOHEMIAN BAROQUE**

In Bohemia the Thirty Year' War concluded in 1620 with the Catholic victory at the White Mountain.

Afterwards, the Czech nobility was forced to leave the country, gradually replaced by the rich Catholic aristocracy coming from different part of the Europe. Along with them, the settlement of the Jesuit order created the basis for an increasing necessity of religious building, leading to an extensive process of construction and reconstruction.

In this context, the Baroque started to spread among the Crown of Bohemia, changing significantly the character of Prague and of all Czech countryside. The political situation along with the peculiar geographic position, resulted in a combination of different styles, overlapping the previous gothic features and leading to the creation of a unique and original style, the Bohemian Baroque (Norberg Schulz, 1968).

The first period, that could be defined as early Baroque, was dominated especially by architects coming from Italy, where the Baroque was born.

In the first half of 17<sup>th</sup> century, the palace was still of particular importance and the first baroque approach started in this kind of buildings. The first in Prague was the Wallenstein Palace (Fig. 2.1), designed by Giovanni Pieroni and Andrea Spezza, with still strong influence coming from Mannerism. This was followed by the Clementinum (1654-1680) of Carlo Lurago (Fig. 2.1), still Mannerist in character but at the end of his career he developed a Baroque architecture, as could be appreciated in the nave of the Cathedral in Passau (1668). The construction of palaces was continued by Francesco Caratti with his Czrenin Palace (1668-1697) (Prochazkova, 2010).



**Fig. 2.1 - Wallenstein Palace; Clementinum (PhotoPrague)**

Carlo Lurago used to work also with the Jesuits, designing several churches as St. Ignatius Church on the Charles Square and the Church in Březnice.

At beginning, the churches were designed with the Italian style without the installation of tower, but the Jesuit Church of Klatovy, designed by another Italian, Domenico Orso de Orsini, could be considered as one of the first attempts of integration with the classic Nordic scheme with two bell towers.

In the passage from Early to High Baroque, a French architect, Jean-Baptiste Mathey (1630-95), had a key role, making the Bohemian Baroque more cosmopolite and with a more refined character. The Church of St. Francis Seraph (Fig. 2.2) could be considered the first monumental central plan building, influencing the successive generation of architects. His works prefigure the High Baroque style in the Czech lands (Norberg Schulz, 1968).

Although the Bohemian High Baroque is a regional style, it represents one of the most suggestive chapter of the European architecture, having been able to develop in an original way the Radical Baroque style created in Italy by Francesco Borromini and Guarino Guarini. Its style is based on the relationship architecture-landscape, both in urban and extra urban contexts, and on plastic closed shapes with internal movements (Norberg Schulz, 1968).

The most important figures of the Czech High Baroque style were Christoph Dientzenhofer, Kilian Ignaz Dientzenhofer along with Jan Blažej Santini-Aichel.



Santini-Aichel, of Italian origin, was involved since the childhood with the father in the works of restoration of the Cathedral of St. Vitus. As a consequence, he developed a personal architecture language called “gothic baroque”, inspired also by the works of Francesco Borromini and Guarino Guarini. His masterpiece, and one of all Baroque, is the Pilgrimage Church of Saint John of Nepomuk (Fig. 2.2).



Fig. 2.2 – St. Francis Seraph (Bell,2008); St. John of Nepomuk (BeyondPrague)

### 2.3. DIENTZENHOFER

Christoph Dientzenhofer (1655-1722) took the concept of Borromini and Guarini, forging them in the Bohemian context. His attempt to merge the thematic of Roman High Baroque with the local traditions resulted in a great use of curved walls, intersection of oval spaces, rhythmical movements determining scenographic effects (Denti, 2001). The masterpieces of Christoph are the Church of St. Joseph in Obořiště, St. Nicholas in Malá Strana (Fig. 2.3), and the Church of St. Margaret in Břevnov.

In all of these, it is possible to appreciate how the architect tried to assimilate the features of Italian baroque.

Christoph's son, Kilian Ignaz Dientzenhofer (1689-1751), was able to complete his path and make even a better and mature fusion of the major architectonic patterns of the period. Likely from the influence of the close Viennese Baroque, he acquired a specific attention about details and decorations, enriching the concept's father of the shaped volume. As Mathey did, Kilian elaborated a peculiar configuration based on the equilibrium of a central space and secondary cells interdependent (Denti, 2001).

The relationship with the context, typical of Baroque, in the Dientzenhofer's works, is extremely important and emphasized. It is interesting to underline the capacity to build up a new cultural landmark, dominating the landscape, as the case of St. Nicholas in Mala Strana, or creating a sort of new scenography in an already existing square, as the case of St. Nicholas (Fig. 2.3) in Stare Mesto (1732-1737, K.D.)

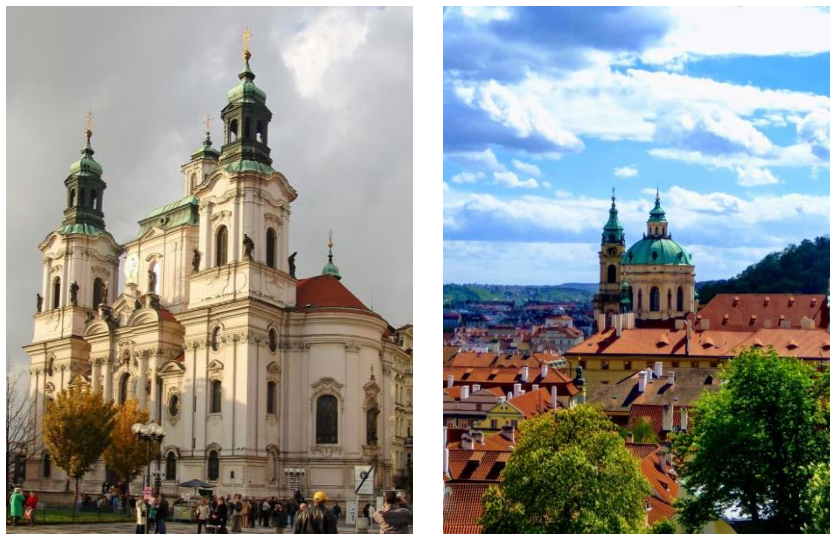


Fig. 2.3 - St. Nicholas in Stare Mesto, St. Nicholas in Mala Strana (Pragl, 2014);

However where the Dientzehofer's production achieved the highest level in terms of integration in the landscape has to be research in the region of Broumov.

## 2.4. BROUMOV REVOLUTION

Broumov Region is a harmonious, ecologically and aesthetically balanced landscape and it is unique in the Czech Republic, for its position, morphology, settlement of civilizations and architecture. Located on the border between Czech Republic and Poland, the region is characterized by an huge diversity, from mountains and valley, to meadows and grassland, with strange bizarre “rock-towns” making the area even more exceptional (Historical Landscapes, 2015).



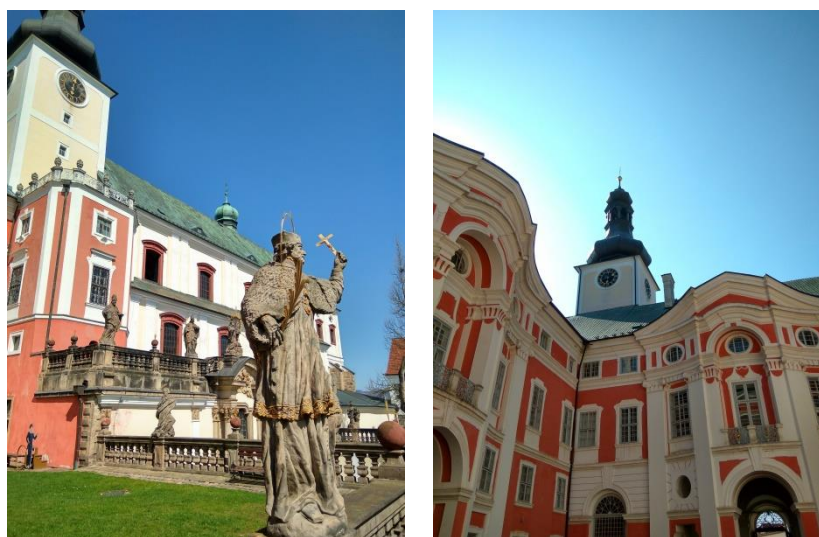
Fig. 2.4 - Broumov Map (Broumovsko)

Immersed in this landscape, several historical buildings are witness of past civilization. In fact, the region started to be colonized in the mid-13<sup>th</sup> century, once been assigned to the Benectine order of Brevnov by King Premysl Otaka I. The towns of Police and Broumov started to be occupied respectively by Bohemians and Germans. Moreover, after the construction of the monasteries, the monks were able to elaborate a plan of colonization, settling several villages with farmland around waterways, with an arrangement that it is possible to appreciate still nowadays.

However the region experienced several political and social conflicts. The mentioned Thirty Years War even exacerbated the already existing condition of conflict (Hussite wars) between the Germans, generally wealthy landlords, and the Bohemians, mostly farmers. After the victory at the battle of White Hill, and as a consequence of the incessant Counter Reformation, the Catholicism reached his peak in Bohemia between the end of the 17<sup>th</sup> century and the beginning of the 18<sup>th</sup> century, involving also the eastern part. The building owned by religious order were taken by Jesuits and Carmelitas, supported by the Catholic aristocracy, creating the conditions for the process of modernization (Cirkl, 2013).

#### BROUMOV MONASTERY

In this process of catholicization, the renewal of the monastery of Brevnov-Broumov was the the first and necessary step, with its exceptional position and the 19 villages that belonged to it. The impulse of reconstruction started from the abbot Thomas Sartorius, who entrusted to the master builders Martin Reiner and Martin Allio, realising a partial rebuilding of the medieval side of the monastery and the construction of the church in Martikovice. However, the responsible of the main activities carried out in the monastery and in the region around, was the new abbot, Omar Zinke, administrator of the abbey for 38 years since November 1700. Meanwhile in Prague, the works of Crisptoph Dientzenhofer had acquired excellent reputation, above all the main nave of the church at Mala Strana. Its splendour convinced Zinke to stop the unsuccessful collaboration with the architect P.I.Bayer and to hire Cristoph, who became in 1709 the permanent builder in Brevnon and Broumov.



**Fig. 2.5 - Terrace in monastery of Broumov; Cloister**



Cristoph designed the scenic terrace of the new access to the church (Fig. 2.5) and its main portal, along with works of reconstruction in the southern and eastern part of the abbey, carried out in a couple of years (Denti, 2001).

The Cristoph's son, Kilian started to be involved since 1916, after his journey abroad, as master builder, and in 1721 he started to work independently, taking the place of the father just one year later. Between the 1727 and 1733, Kilian made completed the reconstruction, building up new wings and adding in the complex the Church rebuilt by Martino Allio. The final result is an astonishing palace, able to combine in itself the elegance of the baroque and the power of a fortress on a hill, dominating all the landscape and villages around (Fig. 2.6).



**Fig. 2.6 – Monastery of Broumov**

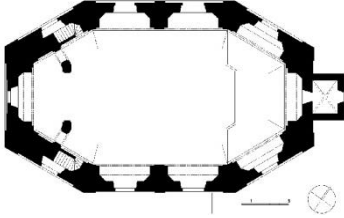

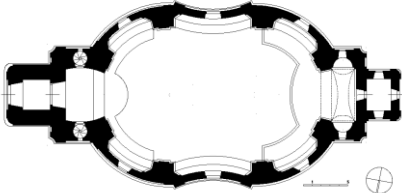

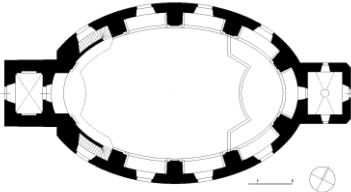

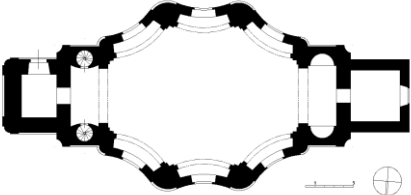

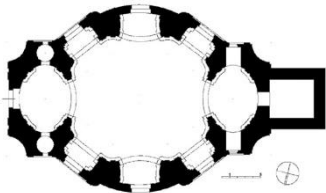

### BROUMOV CHURCHES

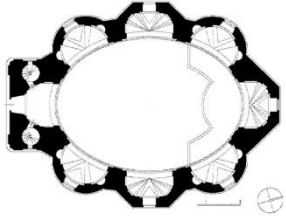

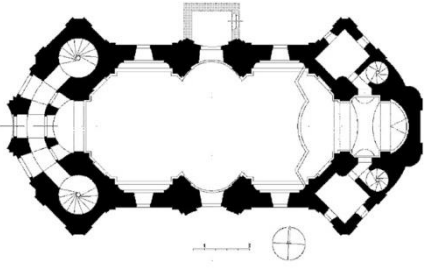

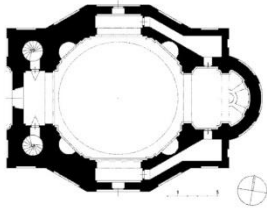
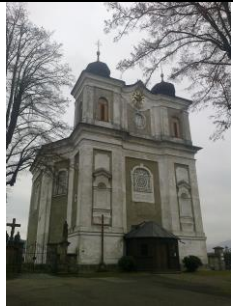
Along with the works of the monastery, the abbot Zinke was aware about the necessity of renovating the network of the old churches. This ambitious programme started from the consideration that keeping the maintenance of the gothic wooden churches would have been too much demanding.

A totally replacing looked as the only option, and the abbot according with the Dientzehofers took advantage of this situation to elaborate one of the most successful complex of Baroque buildings in Bohemia (Prokop, Kotalík, & Suva, 2001).

Almost the totality of the churches were built by Kilian, even if the churches in Vernerovice (1719-1726), in Ruprechtice (1720-1730) and in Octovice (1725-1726) had been designed by the father. On the other hand the following churches were designed and built by Kilian: in Hermankovice (1722-24), in Viznov (1724-25), Bezdekov (1724-27), Sonov (1726) and in Bozanov (1735-43).

## OVERVIEW CHURCHES

<p><b>Church of St. Jacob The Bigger, Ruprechtice</b></p>		
<p><b>Church of St. Anna, Vižňov</b></p>		
<p><b>Church of St. Michael, Verněřovice</b></p>		 <p>(Novak, 2010)</p>
<p><b>Church of all saints, Heřmánkovice</b></p>		 <p>(Baranek, 2008)</p>
<p><b>Church of St. Mary Magdalene, Božanov</b></p>		 <p>(Kostelycz)</p>

<p><b>Church of st. Barbara, Otovice</b></p>		
<p><b>Church of St. Margaret, Šonov</b></p>		 <p>(Spelda)</p>
<p><b>Church of St. Prokop, Bezdekov</b></p>		

From the overview of plans it is easy to notice the difference in the style and the approach between son and father. Christoph, at the end of his career, did not try specific innovations and kept designing the plan with the already tested centralized oval or octagonal system, with embedded pillars. The son carried out a modification of this basic scheme, a combination of a longitudinal and central plan, solving the problem of the depth perception.

In many cases, the Dientzehofers were allowed to choose independently the location of the churches, a very uncommon opportunity for any architects.

This chance, along with a deep knowledge of the countryside, made possible the planning of a perfect integration between the churches and the landscape. Each church was built as new landmarks, with a dominating position on the developing villages, even recognisable from great distances. This feeling was emphasized by the mutual visual connection between the churches and towards the monastery as well, representing a marvellous global design of the whole region.

The great skills of the Dientzehofers were not only under the architectural point of view. In fact, they were able to combine several and difficult requirements and restrictions, in a smart engineering process.

The specific function of country churches led the architects to elaborate a simple design of a single nave with a bell-tower. Moreover, the limitations of funding made necessary the implementation of some expedients in order to keep cheap, fast and easy the process of construction. As the solidity of the construction was a fundamental aspect, and the building up of vaults or domes need expert masons and high funds, the totality of the churches, except one, had false wooden ceilings. The faults vaults were supported by the above wooden roof, very distinctive feature of the Broumov' churches with its steep hip and red painting. This escamotage allowed to arise walls of smaller thickness and to solve, cleverly, static and climatic issues, ensuring uniform loading. Moreover, the skills of the craftsmen on timber construction were more advanced and so also cheaper (Prokop, Kotalík, & Suva, 2001).

With the exception of the Church of St. Mary Magdalene, the other churches were built up only in ten years, from 1719 to 1729.

#### CURRENT SITUATION

The 19<sup>th</sup> century was witness of a great prosperity of the region due to the development of textile industry and the political freedom, allowing also commercial trading. The number of inhabitants within the villages increased reaching almost 1500 citizens for each.

The situation was completely reversed in the following century. After World War II the inhabitants of German origin, that refused citizenship of Czechoslovakia, were forced to leave the country, leading to a progressive demographic decline, along with the abandonment of industries and agriculture.

The communist ideology involved the imposition of atheism, affecting the use and maintenance of religious buildings. Basically, the cultural, social and industrial continuation of the region was stopped, arising the prosperity experienced between 17<sup>th</sup> and 18<sup>th</sup> century (Historical Landscapes, 2015).





### 3 ST. JACOB CHURCH

#### 3.1. ARCHITECTURAL OVERVIEW

The church of St. Jacob Greater was built in the years 1720 - 1723 on the site of the older wooden gothic church, designed by Christian and concluded by Kilian Dientzenhofer.

The Church is located in the village of Ruprechtice, in the north part of Broumov region. As the first written records (1361) report, the village was founded in the middle of 13<sup>th</sup> century by the German settlers, forming a gathering of houses along the creek below the highest peak of the Javoří Mountains(Fig. 3.1).

The location of the church, on an slight hill, was probably selected in order to create a sort of main entrance to the village (Fig. 3.2), at the corner of the roads coming from the city of Mezmești and Broumov (7 km distant).



Fig. 3.1 - Drawings of Ruprechtice of 1676; Map of Ruprechtice of 1780 (Historical Landscapes)



Fig. 3.2 - View of the facade from the main road; Aerial view (Google Maps)

So Christoph Dientzehofers to emphasize this purpose, elaborated a monumental and austere façade, with an orientation at SW, creating a landmark able to “introduce” the village behind (Fig. 3.2). The façade consists of two level. The first very simple, flat and characterized by the portal and the arch window along the principal axe, with small opening and relevant arches at the sides. The second level is a sort of shaped gable with a curve cornice, smoothly linked to the level below by two spirals (Fig. 3.3). This configuration started in Italy by Giovan Battista Alberti in the Church of St. Maria Novella and widely used by the Jesuits. Among the Broumov’ group of churches, St. Jacob is the only one having this peculiarity, not presenting one or two bell towers on the main façade, as usual in the Bohemian Baroque. In fact, the bell tower was added in the late 16<sup>th</sup> century, as a free-standing structure at the NE of the complex, creating a further access to the cemetery surrounding the church. However, in the previous works of Christian it is possible to find a similarity of this kind of facade in the complex of Broumov monastery ( Fig. 3.3).



Fig. 3.3 - Main façade of St. Jacob; Façade in the complex of the Broumov Monastery

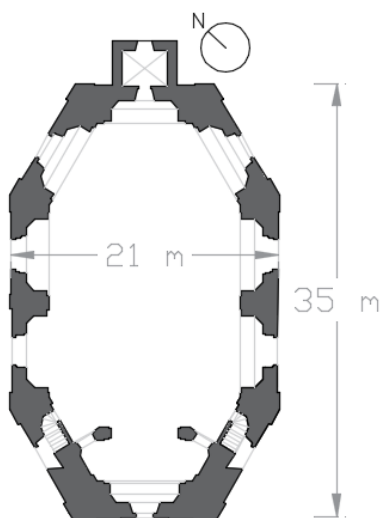


Fig. 3.4 – Plant of the Church

As it is typical of Baroque, the main façade is not anymore a natural continuation of the internal distribution. In fact, the church consists in a central space shaped as an prolonged octagon (35m x 21m). Externally, the walls (15 m high) are characterized by slight recesses, creating a sort of frame surrounding the arch-window. The windows follow a regular distribution, and correspond to the arcade niches on the internal perimeter, which ensure a rhythmic movement of the space distribution.

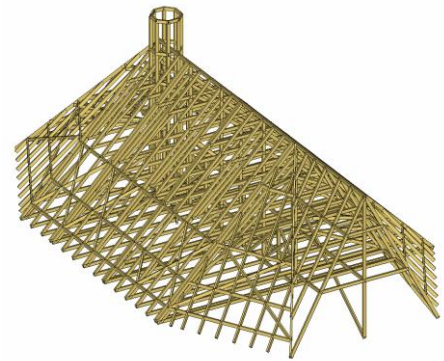
On the first floor of the niches there is a gallery with a wooden balustrade, creating a sort of “matroneo” (Fig. 3.7), feature typical of Romanic, along with an organ located upon the main entrance. The structure is completed by the square cross-vault sacristy, and



by the entrance on the SE side, added in the last century.

The internal space is perfectly and uniformly closed by the wooden false vaults, that rests on the continuous cornice, carried out by the pillars and the walls.

The configuration of the church is completed by the timber roof, with its peculiar shape. For the impossibility to have access to the roof, it is necessary to use as reference the work done in her thesis by Giulia Facelli. As it is possible to notice from the drawing (Fig.3.5.), the configuration is extremely complicate, involving trusses and beams with different functions, as the support of the roof, of the platform and the wooden ceiling (Facelli, 2014).



**Fig. 3.5 3D Drawing roof (Facelli,2014)**



**Fig. 3.6 - South point of view of St. Jacob; East point of view (Susanti, 2017)**



**Fig. 3.7 - Internal view of the main altar; Internal view of matroneo (KHK)**

### 3.2. MATERIAL OVERVIEW

After the geometric and architectonic description, it is important to define the material features of the building.

#### Stones

The church was built, as the rural nature of the project required, with the stones available in the surrounding area. Broumov region is rich in quarries, with an high prevalence of sandstones. There is evidence, since the beginning of 18<sup>th</sup> century, of quarries located in Bozanov and Vernerovice.



**Fig. 3.8 - Geological Map of Broumov Region; Picture of the quarry in Bozanov (Lom Bozanov, 2010)**

The first one provided an huge amount of material for constructions overall the Czech Republic, having very good physical-mechanical properties, especially strength, frost resistance, absorbency and abrasion resistance. Macroscopic aspect varied from light grey to beige, showing sometimes a darker, rusty brown colour (Vavra & Stelcl, 2010).

The sandstones coming from Vernerovice are distinguished for a peculiar reddish-brown colour. Moreover, it is noticeable the presence of quarries very close to the location of the church, in Ruprechtice, Jetrichov and Mezimesti. In this last case, the macroscopic appearance is as well reddish-brown or rusty brown, but the structure is strongly porous. In Jetrichov the appearance is more variable involving grey, violet-grey, pinkish or brownish arctic sandstone, but incoherent, and easily disintegrating to sand (Vavra & Stelcl, 2010).

This brief overview of the surrounding quarries is useful to well understand the reason of the great variability of stones in the arrangement of the church 'walls. Moreover the different characteristic could help in finding the reasons of the different deterioration observable in the external facades.

In fact, being formed from the sedimentation of accumulated detrital, the characteristics of sandstones depend by several factors connected to the nature of the detrital material and the environment of deposition. The weathering resistance is strongly influenced by the poor size distribution, texture of the rock, the mineral composition and cementation type. The effective porosity is a condition of potential for the stone to take in and hold water, and hence to weather (Labus & Bochen, 2012). Taking into account that porosities could vary in a range from 0 to 35%, it is easy to find the reason of such differential deterioration for stones subjected to the same external actions.

In the arrangements, less often, igneous stones as Ignibrite occurred, along with other layered sandstones formation as sandy shale. A campaign of investigation has been carried out, by means of a Schmidt Hammer, in order to identify the mechanical properties of the different stones.

#### Rendering

From the point of view of the external render covering the external walls surface, two different typologies can be detected. The first one, very thin, made of lime mortar, applied on the “frame” of each walls, and the second one, thicker, covering the central part of each wall, made of large sand aggregates.

#### Roof and ceiling

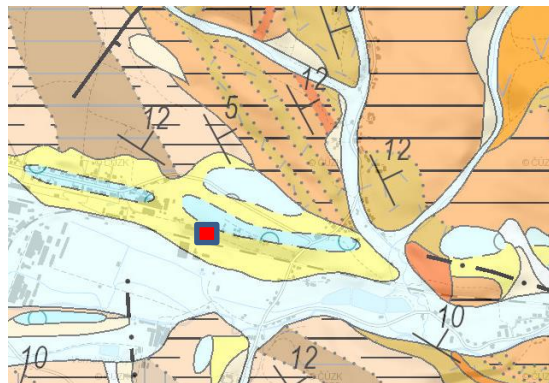
As mentioned in the previous chapter, the roof and the ceiling are made completely of timber. The ceiling, arranged with timber boards, is covered by gypsum and lime plaster.

The structure is made basically by prunes, covered two different materials: stone tiles and metal sheets. These last have been probably a cheap intervention in order to repair several points of the roof (Facelli, 2014).

### **3.3. SOIL OVERVIEW**

Being the condition of the soil a source of concern for the future stability of the church along with the current damages, it is important to provide information about the area surrounding the church.

As defined in the geological map (Fig. 3.9), even if the Broumov region is mainly characterized by sedimentary sandstones and rhyolite (Palaeozoic era), the church is founded on a relatively recent soil region (Mesozoic era), made of soft clay and loamy clay, with fluvial origin (Ceska Geologicka Sluzba).



**Fig. 3.9 - Detail of geological map in Ruprechtice (Ceska Geologicka Sluzba)**

### **3.4. DAMAGE OVERVIEW**

Before to proceed with a detailed description of the damages of the St. Jacob Church, it would be useful a general overview of the current situation, providing also a comparison with other churches of the same region, when available.

The damages observed during the visual inspection are linked to a lack of maintenance of the church since the Second World War, which events, as mentioned, left the Broumov Region under the



Communist regime. In fact, several damages experienced by the churches are strictly related to problems of water penetration, to the high moisture content in the walls and to the phenomena of rising capillarity. The most severe consequences of this situation is the formation of biological colonization on the interior enclosure walls and the loss of plaster's integrity and of the stones as well. Externally, the thinner render results to be loss in almost the totality of the surface, leaving the stones subjected to the external actions. The thicker render, as well, is missing in different location, being characterized by phenomena of detachment and bulging. This kind of damages are peculiar of the churches investigated, the one discussed in this work of thesis, St. Jacob, but also St. Anna and St. Barbara (Fig. 3.10).



**Fig. 3.10 – Deterioration of the render in St. Jacob Church (Left); St. Anna (Central); St. Barbara (Right)**

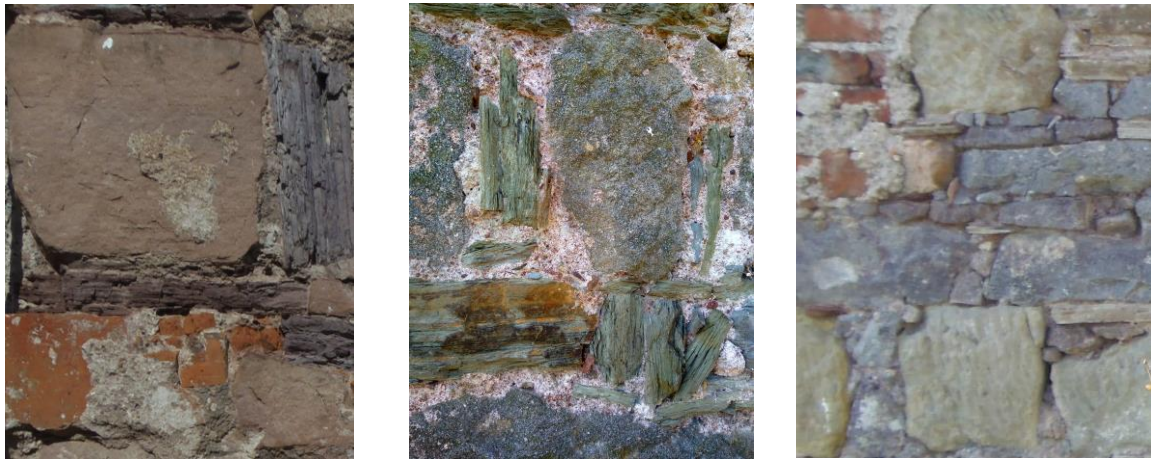
The reason of this common situation has, as mentioned, to be related to the presence of water, coming from the soil and from the roof for the poor condition of the gutters (Fig. 3.11). As it is noticeable from the picture, the soil, in correspondence of not working gutters, appears deteriorated and could have a role in settlement phenomena.



**Fig. 3.11 – Deteriorated soil; Detail of the gutters**

This scenario involved phenomena of efflorescence, leading to plaster detachment, and phenomena of freeze-thaw cycles of the stones. As already discussed above, the differential degradation of stones

spread all over the walls' surface as the use of stones of different features (Fig. 3.12). As this approach was used also for the other churches, it is possible to find the same kind of damage in St. Barbara and St. Anna (Fig. 3.12).



**Fig. 3.12 - Differential deterioration in St. Jacob (Left); St. Barbara (Central); St. Anna (Right)**

The high level of moisture, along with lack of ventilation, had negative impact also in the interior of the Church, involving peeling, discoloration and biological colonization on the surface of the walls. The walls most affected are the ones oriented to NW, being under the shadow of the trees and subjected directly to the sunlight from the opposite windows, ensuring the ideal situation for the biological growth (Fig. 3.13).



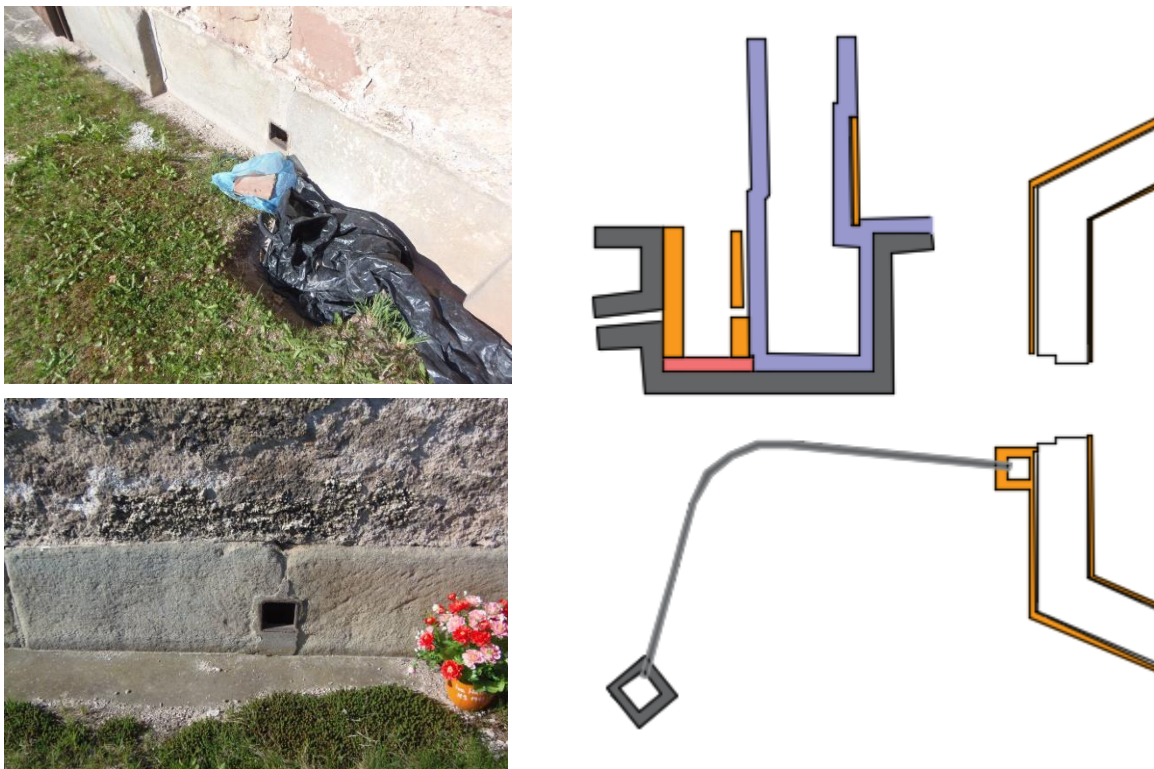
**Fig. 3.13 - Rising dampness and Biological colonization on the internal walls**

Also the other churches experienced similar damages, in St.Barbara and even more in St. Anna. Especially this last has been subjected to rainwater flow coming from the adjacent slope, involving severe issues also of soil settlement.

The situation in St. Jacob is slightly better, under this point of view, as it is built upon a slight hill, avoiding the accumulation of water on a single side of the church.

Moreover, in the '40 some works were carried out to improve the overall situation, even if its efficacy could be put in discussion. A drainage system was installed around all the perimeter in order to collect the water and decrease the problem of rising dampness (Fig. 3.13). This past intervention will be considered again under another point of view in the chapter dedicated to settlement analysis.





**Fig. 3.14 - Drawing and detail of the drainage system built in 1937**

Along with the problems related to the moisture, during the survey, several cracks were detected in the walls and in the ceiling.

The reasons are different and it is one of the goal of the thesis to understand the behaviour of the structure and define which cracks are due to external actions.

The cracks of the ceiling have to be related to the false vault system used in the Broumov group of churches. The cracks occurred, in fact, along the above main trusses, caused by the long term action and deformation of timber elements (Fig. 3.15). The same configuration of damage has been detected also in St. Barbara. (Fig. 3.15)



**Fig. 3.15 - Cracking pattern in the ceiling of St. Jacob (Dienzenhofer Adventures); Cracks in the ceiling of St. Barbara**



The majority cracks on the walls do not represent source of wondering. Some of these are related to some concentration of stress (Fig. 3.16.a) due to local punctual load. The cracks at the corner (Fig. 3.16.) of the walls could be linked to discontinuity of the shape. Minimal cracks occurred in the top part of the openings (Fig. 3.16.c), and could be due to normal behaviour of arches under self-load or could be the first signs of other phenomena as it will be explained after the 3D analysis. Some minimal cracks seem to involve just the plaster and this could be related even to the level of moisture and salt concentration in the stones below (Fig. 3.16.d).

Source of wondering could be the cracks located in the niche at north (Fig. 3.16.f) and in the vault of the sacristy (Fig. 3.16.e). These cracks can be related to a not dangerous formation of relieving arches due to the presence of openings. However, they could be connected to soil settlements phenomena, hypothesis that will be verified in chapter 6.


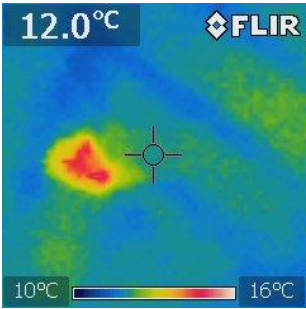
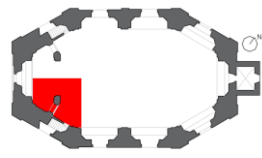


**Fig. 3.16 - Details of cracking pattern in the walls of St. Jacob**



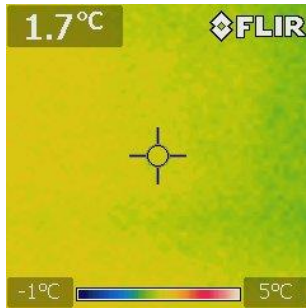
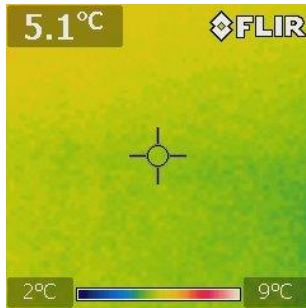
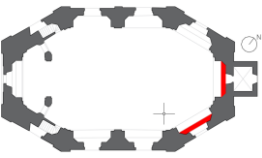
### 3.5. DETAILED DAMAGES IDENTIFICATION

A detailed identification of the damages observed during the inspection is provided in the tables below. For any kind of damages an example is showed, the majority of the tables are available in the Appendix.


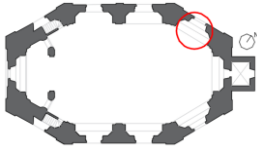
Tab. 3.1

DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Hole in the wood ceiling</b>	Due to water infiltrations from the roof, a small portion of the ceiling is completely lost.		
LOCATION			
			


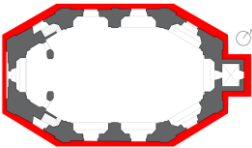
Tab. 3.2

DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Rising dampness</b>	The walls at east and north-east experienced rising dampness, until a height of 2 m. This involve deterioration and loosing of the render.	 	 
LOCATION			
			

Tab. 3.3

DAMAGE	DESCRIPTION	PHOTO
<p><b>Cracks upon and above the arch in the cell</b></p>	<p>An horizontal crack characterizes the niche located at the north of the church.</p> <p>The reasons for causing this damage could be different, and probably related to soil settlements.</p>	
LOCATION		
		

Tab. 3.4

DAMAGE	DESCRIPTION	PHOTO
<p><b>Bulging of Render</b></p>	<p>For reasons related to moisture and salt crystallization on stone surface, the render experienced phenomena of bulging, reaching in some locations values of several cm.</p>	
LOCATION		
		



#### 4. BEARING CAPACITY OF THE ENCLOSURE WALLS

As presented in the introduction, the first goal of the thesis is the analysis of the bearing capacity of the enclosure walls. Among the Broumov's churches this aspect represents an important uncertain coming from the peculiar typology of walls to be faced in this case study. In fact, the not regular arrangement of the stones and the use of different materials does not allow an easy application of standards, as Italian code or similar approaches. So, it is necessary a more sophisticated analysis by means of micro modelling. Afterwards, the goal is the application of the derived properties in the 3D model that will be presented in the following chapters.

In order to carry out this task, it is fundamental a detailed information of the geometry and material properties. Taking advantage of the current situation of render's deterioration, it was possible to collect two different portions of the walls, identifying the textures. The locations, in the main façade and in the NW longitudinal wall, were selected as the most representative of the church in terms of arrangement and stone used (Fig. 4.1).

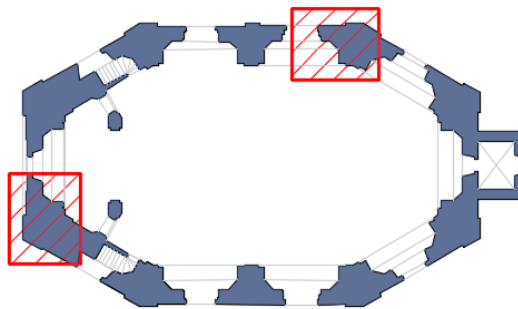


Fig. 4.1 - Locations of the Schimdt Hammer tests



Fig. 4.2 – Walls investigated



#### 4.1. SCHMIDT HAMMER CAMPAIGN

As mentioned in the material overview, a campaign by means of Schmidt Hammer was carried out in order to get the compressive strength of the different stones. For a better understanding of the arrangement, a detailed drawing of the walls is proposed below, with the investigated stones marked with a number (Fig. 4.3).

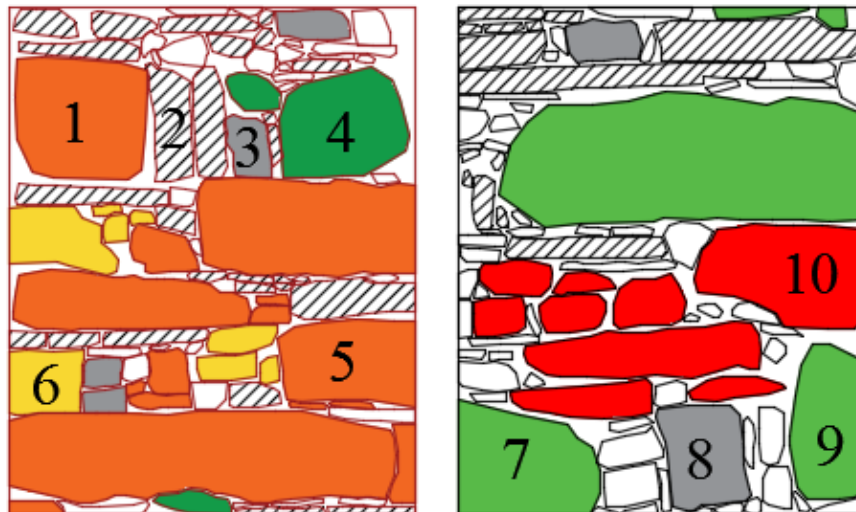


Fig. 4.3 – AutoCAD drawings of the walls

Similar colours were used to highlight stones having a similar geotechnical origin, red- brownish sandstones (in red and orange), green sandstone (in green), bricks (in yellow), Ignimbrite (in grey), deteriorated stones (sketched). The test on the deteriorated stones was not possible as the ruined surface that made the Schmidt Hammer's result not reliable.

Tab. 4.1 – Results for any stones investigated with the Schimdt Hammer

	Q1	Q2	Q3	Q4	Q5	Q6	Q7	Q8	Q9	Q10
	50.5	17.5	38.5	33.5	42.0	46.0	18.5	55.0	30.0	36.5
	42.0		30.5	29.5	39.5	46.5	14.0	57.0	36.5	42.5
	57.5		36.0	33.0	47.0	38.5	29.0	46.0	26.0	65.0
	45.5		32.0	20.5	46.5	39.5	17.0	55.0	20.5	56.5
	56.0		45.5	28.0	46.0	43.0	32.0	48.5	33.0	52.5
	51.5		34.5	38.0	46.0	39.5	31.5	40.0	26.5	47.5
	48.0		22.5	25.0	54.5	46.5	27.0	46.5	26.0	40.5
	64.5		32.0	29.5	46.0	46.0	28.5	42.0	24.0	44.5
	47.5				47.0	45.5	19.5		29.5	60.5
	48.0				40.0	44.0	25.5		34.0	62.0
Average Q	51.1	17.5	33.9	29.6	45.5	43.5	24.3	48.8	28.6	50.8
$f_{ck}$ (MPa)	39.6	7.2	20.0	16.1	32.5	30.2	11.8	36.6	15.2	39.2

The previous table contain the results obtained during the investigation on site. It has to take into account that the values got from the Schimdt Hammer were converted, through an empirical formula, in the values of compressive strength of the stones.

$$f_c = 0.0108 * Q^2 + 0.223 * Q$$

These values have to be evaluated carefully as very sensitive to the condition of the stone surface, and in presence of moisture. However, they played an important role in detecting the different properties of the stones, in terms of compressive strength, making possible a comparison with the values coming from literature (Čáchová, Koňáková, Kočí, & Vejmelková, 2018). The definition of the other material properties are explained in the following chapter.

#### 4.2. MATERIAL PROPERTIES FOR MODELLING

The modelling of the walls was carried out through the software ATENA 2D, analysing separately the models of the longitudinal wall and the sectional walls. ATENA is a software dedicated to reinforced concrete structure, and the application to masonry has to be treat carefully. The constitutive model selected has been the “Non linear cementitious 2” (Cervenka & Jendele, 2018). In this model, it is settled for tensile behaviour an exponential opening law (Fig. 4.4), where is necessary to set directly the fracture energy in tension.

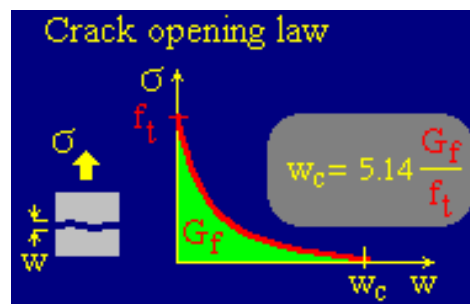


Fig. 4.4 – Exponential opening law (ATENA Manual)

For compression, the strain at the peak stress has to be defined, along with the critical displacement  $w_d$  that defines the slope of the linearly descending branch after peak stress (Fig. 4.5).

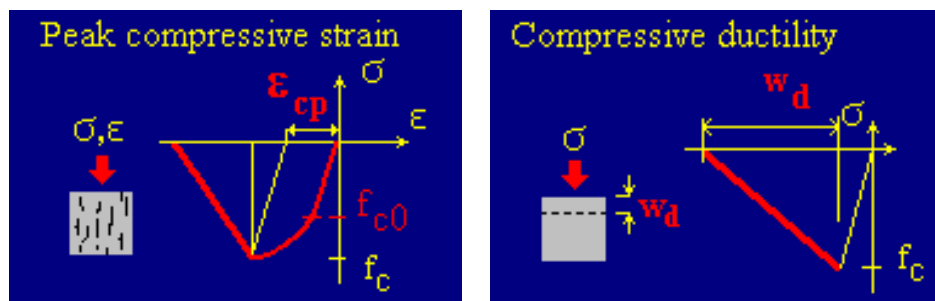


Fig. 4.5 Peak compressive strain and critical displacement in compression (ATENA Manual)

The definition of the material properties involved several considerations. The compressive strength was defined accordingly with the values obtained from the Schimdt Hammer, selecting the minimal

value of the same typology of stones. Regarding the modulus of Young, a value in literature for red-brownish sandstone was selected (Reiterman & Holcapek, 2015). For the restant sandstones, taking into account that typically the range is between 5-25 GPa, the elastic modulus was calculated in relationship with the compressive strenght, with the exception of the degraded stone where a reduced value was considered. The values of tensile strenght were considered in a range of 1/10 and 1/20 of the compressive strenght. From these values, fracture energy in tension was obtained by means of ductility index parameters  $d$  taken equal to 0,029 mm (Lourenço, 2008).

$$t_f = \frac{1}{10} \div \frac{1}{20} f_c \quad G_f = d * t_f$$

To define the compressive softening, the peak strain was computed from the compressive strenght and the elastic modulus. The critical displacement is set by Atena for concrete equal to 0,5 mm that is a value still reliable for stones. For mortar was considered one order more, to consider the major ductility. The restant parameters for mortar and rubble are taken from literature (Tarque, Benedetti, Camata, & Spacone, 2014) and Italian Code (*Table C8A.2.1*, Circolare 617, 2009), taking into account that, during the visual inspection, the mortar resulted very weak.

The table below synthetizes the values for any materials modelled in the walls.

**Tab. 4.2 Properties of the materials**

	[Gpa]		[Mpa]		[N/m]		[kN/m3]
	E	v	fc	ft	Gt	$\epsilon_c$	$\rho$
Red sandstone	20.0	0.20	30.0	1.50	43.5	0.0015	21
Green sandstone	8.0	0.20	12.0	1.20	34.8	0.0015	21
Grey Sandstone	13.0	0.20	20.0	2.00	58.0	0.0015	21
Bricks	5.0	0.20	18.0	1.80	52.2	0.0036	19
Degraded	7.0	0.20	20.0	1.00	20.0	0.0029	19
Mortar	0.1	0.25	2.5	0.05	10.0	0.0217	20
Rubble	0.7	0.20	2.0	0.05	10.0	0.0029	20

Two different approaches were carried out, studying the capacity of the longitudinal and transversal section of the walls separately.

### 4.3. LONGITUDINAL WALL

In the longitudinal models, the arrangement of the stones set out in Atena results extremely realistic, with some simplification in order to avoid problems at the mesh stage. The dimensions of the model are 1m x 1.25 m and the mesh size is set approximately around 1 cm, as the average thickness of the mortar (Fig. 4.6). At the base and at the bottom of the “specimen” two steel plates were modelled in order to redistribute equally the load applied, which consists in a imposed displacement of 0,5 mm for each step. In this case the hypothesis of plane stress are fully respected and as the limited dimensions, if compared with the entire wall, horizontal simple supports were applied at the two lateral edges. In the next pages, the model with mesh and the results in terms of vertical stress and distribution of cracks of the two longitudinal walls are provided (Fig. 4.7).



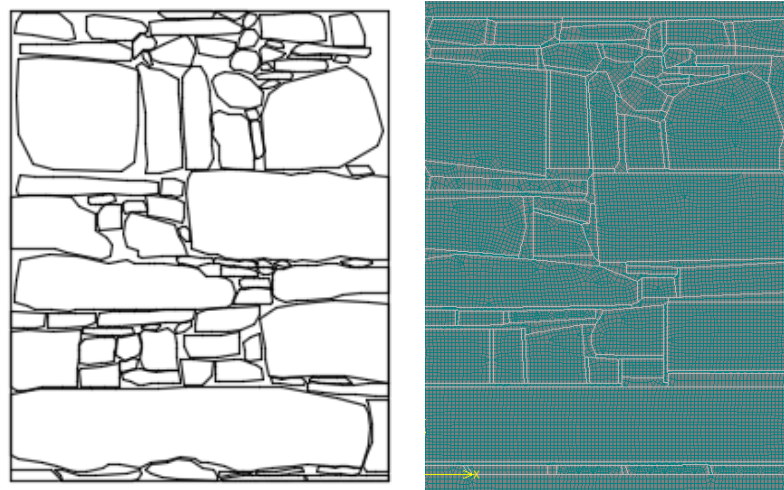


Fig. 4.6 – Realistic drawing and Simplified model in ATENA for Longitudinal Wall 1

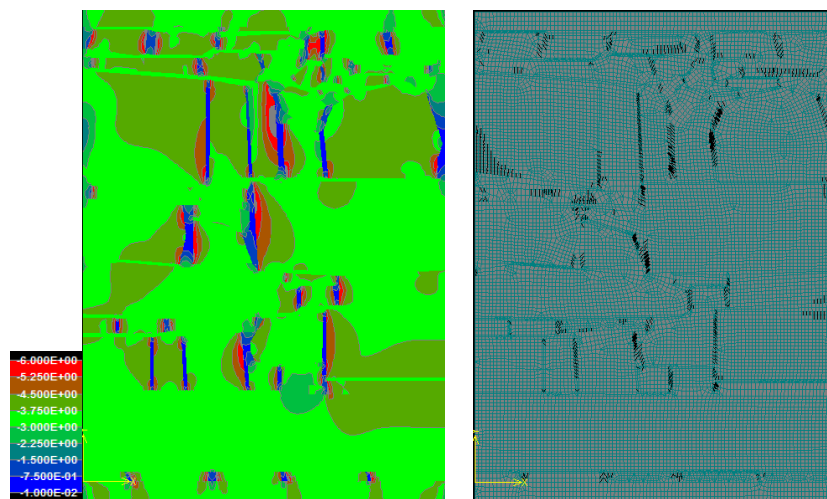


Fig. 4.7 - Distribution of vertical Stress  $\sigma_z$  and Cracking pattern for Longitudinal Wall 1

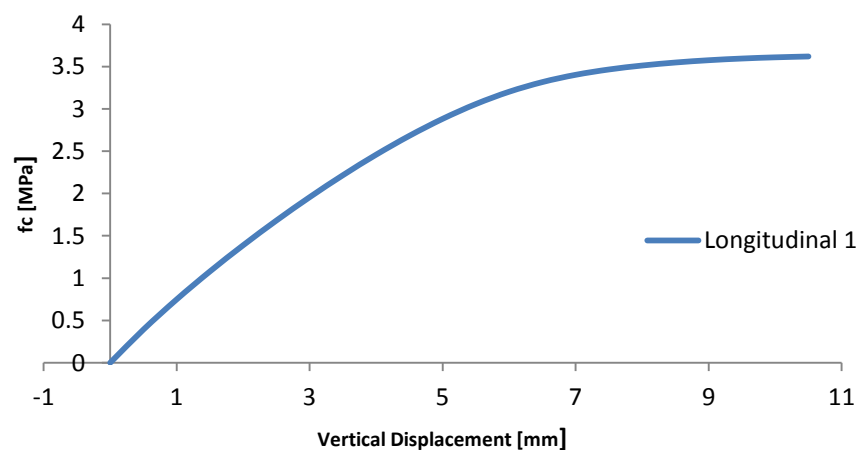


Fig. 4.8 – Curve Displacement – Vertical stress for Longitudinal Wall 1

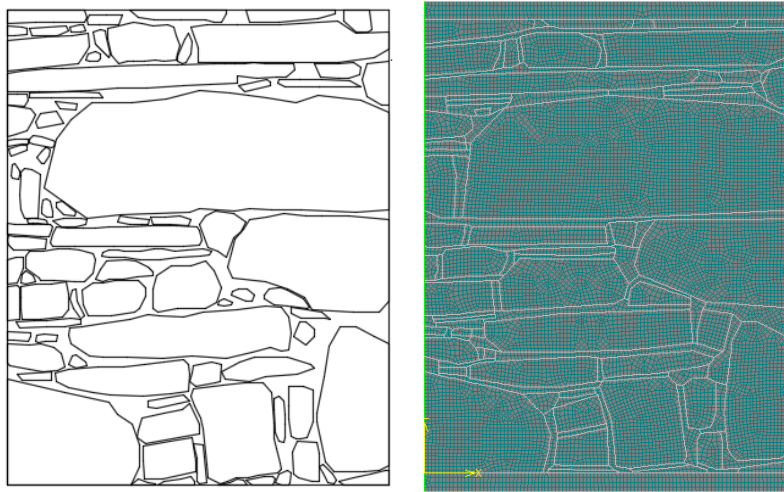


Fig. 4.9 - Realistic drawing and Simplified model in ATENA for Longitudinal Wall 2

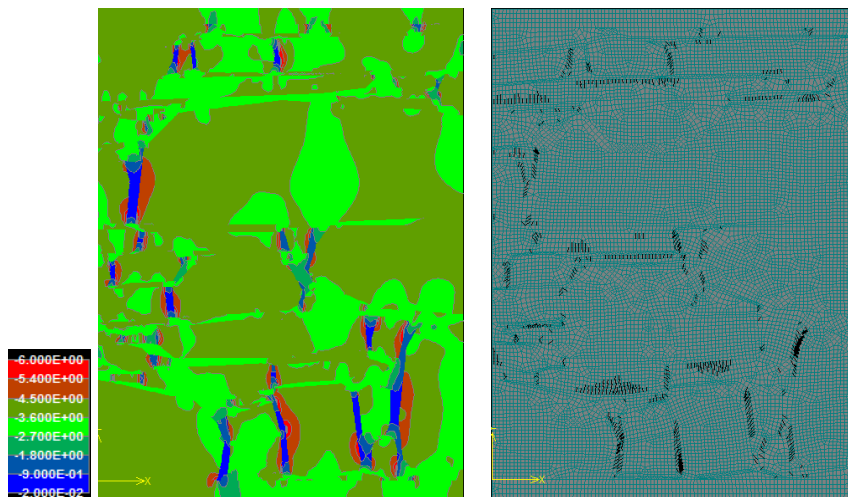


Fig. 4.10 - Distribution of vertical Stress  $\sigma_z$  and Cracking pattern for Longitudinal Wall 2

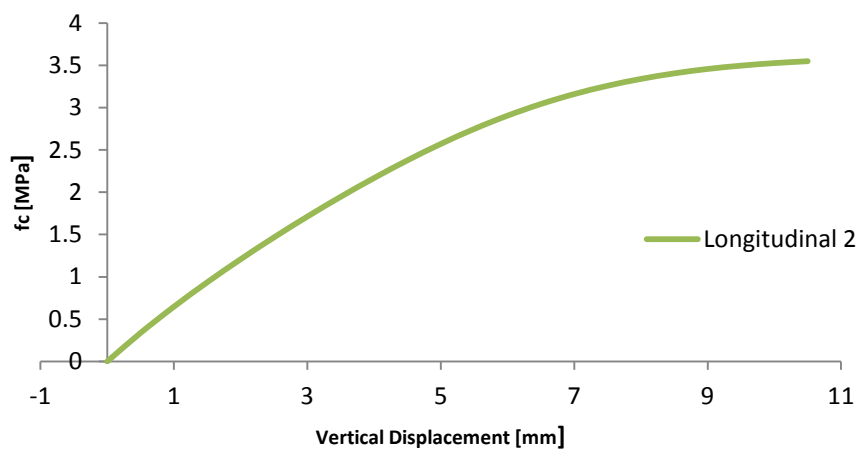


Fig. 4.11 - Curve Displacement – Vertical stress for Longitudinal Wall 2

As it could be appreciated in the previous pictures the behaviour under compression is pretty much similar, with the cracks occurring along the line of mortar, and a distribution of vertical stress that is comparable. Both the models show a peak of compressive stress in correspondence of the plateau equals more and less to 3,5 MPa, with a tangential elastic modulus around 0,8-1,0 GPa (Fig. 4.12).

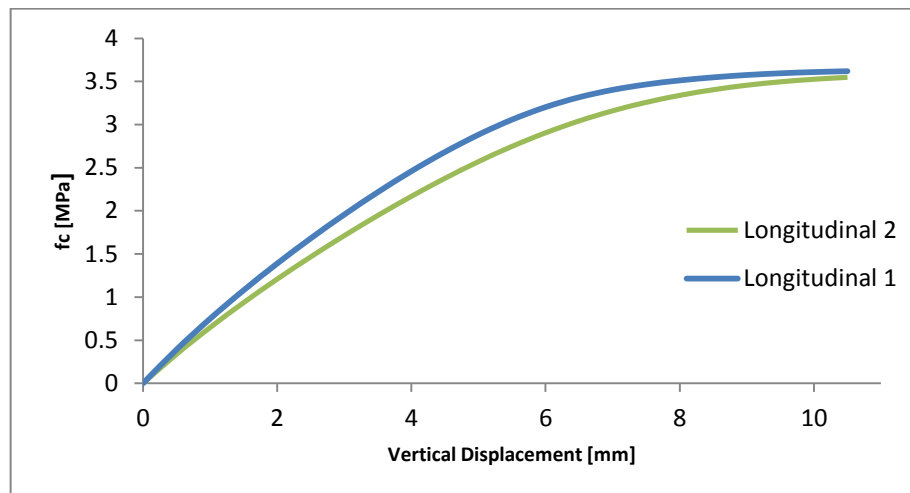


Fig. 4.12 – Comparison of the results

Tab. 4.3 – Results coming from the compressive tests

	Compressive Stress (MPa)	Tangential Modulus (GPa)
<b>Longitudinal Wall</b>		
<b>1</b>	3,60	0,95
<b>Longitudinal Wall</b>		
<b>2</b>	3,55	0,81

#### 4.4. SECTIONAL WALL

Along with the models of longitudinal sections, several model of transversal sections were analysed. In this case the uncertain that characterized the study are much more than the case presented above. From past inspection is known that the enclosure walls, in correspondence of the niches where the thickness is 1.2 m, is made of three leafs, with the internal leaf made of rubble and approximately 40 cm thick. However, the internal distribution and the internal shape of the stones is unknown, and the effectively material of rubble inside the wall has never been investigated.

For both locations already explored for longitudinal sections, three different sectional walls were modelled, simulating: - a very simple disposition and almost unrealistic of the stones, - an irregular internal surface, - the presence of internal connections (Fig. 4.13).

The dimension of the “specimens” in this case is about 1.2m\*3.8m, with a mesh size of at least 1 cm and under hypothesis of plain strain. The constitutive mode, already used, “Cementious Non Linear” is

able to take into account this hypothesis, making modification of Young's modulus and Poisson's Modulus. A steel plate was applied on the top of the model in order to redistribute the imposed displacement. In this case the fixed support is only at the bottom of the wall.

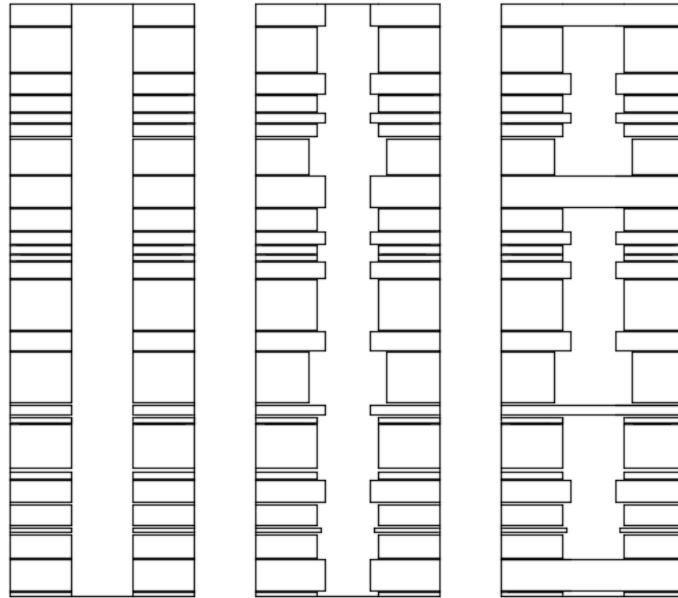


Fig. 4.13 – Drawings of Sectional Wall 1, Simple (Left); Interlocking (Central); Connected (Right)

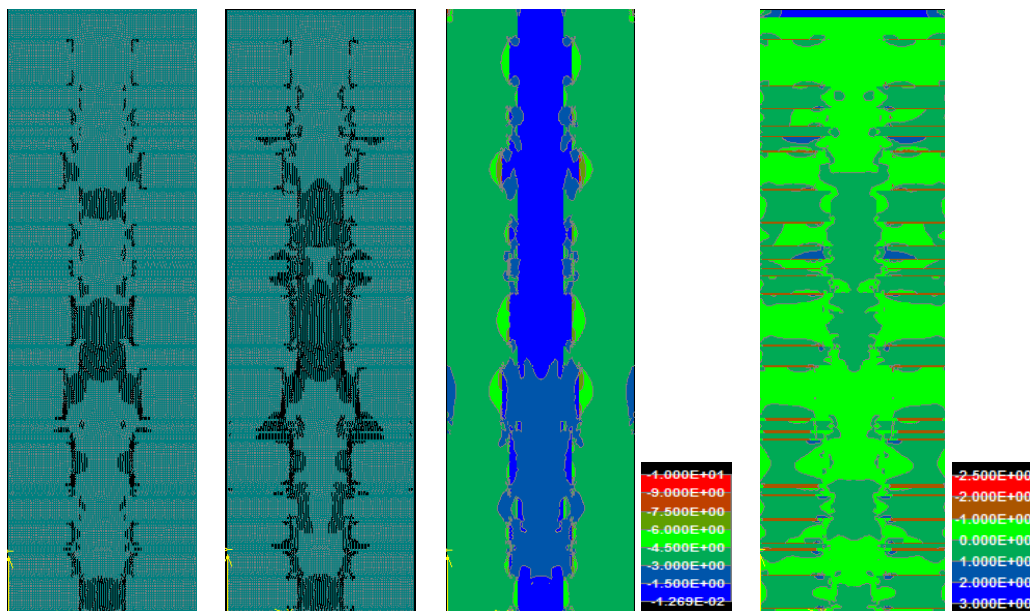


Fig. 4.14 – Cracking pattern of Interlocking Model 1 (5<sup>th</sup> and last step) , Vertical  $\sigma_z$  and Horizontal  $\sigma_x$  stress distribution

Concerning the model b), the pictures above describe the distribution of cracks in two different steps, and the distribution of vertical and horizontal stress (MPa) at the last stage explored. The cracks started to occur at the interface between stones and rubble, to spread, afterwards, within all the

internal leaf. As expected the vertical stress is higher at the interface, and more concentrated in the external leaves, with the rubble carrying a low part of the load applied.

As it is possible to notice in the graph (Fig. 4.15), in this case it resulted much more difficult to get the plastic branch of behaviour in compression. Probably, this is due to the deeply difference in stiffness between the external leaves, made of stones, and the internal rubble. However, the result preserved its importance in the definition of elastic modulus and in the maximum compressive stress before to enter in the global plastic field. In order to have better information, it would be necessary the modelling of the interface between the materials, allowing the detachment and a more realistic behaviour in the case of sectional walls.

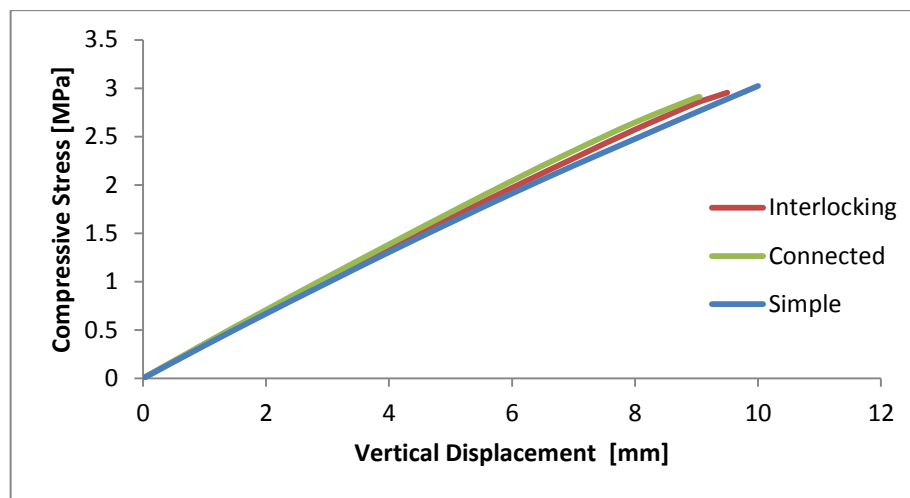


Fig. 4.15 - Comparison curves Displacement – Vertical Stress of the Sectional Walls 1

Tab. 4.4 - Results coming from the compressive tests

	Compressive Stress (MPa)	Elastic Modulus (GPa)
<b>Sectional Wall Simple 1</b>	3,25	1,30
<b>Sectional Wall Interlocking 1</b>	2,95	1,35
<b>Sectional Wall Connection 1</b>	2,92	1,40

The results of the models are very close each other, with a maximum compressive stress of 3 Mpa and an elastic modulus around 1,3 GPa.

The same procedure was carried out for the wall in the second location, at NW of the church, obtaining similar results as noticeable in the next page.

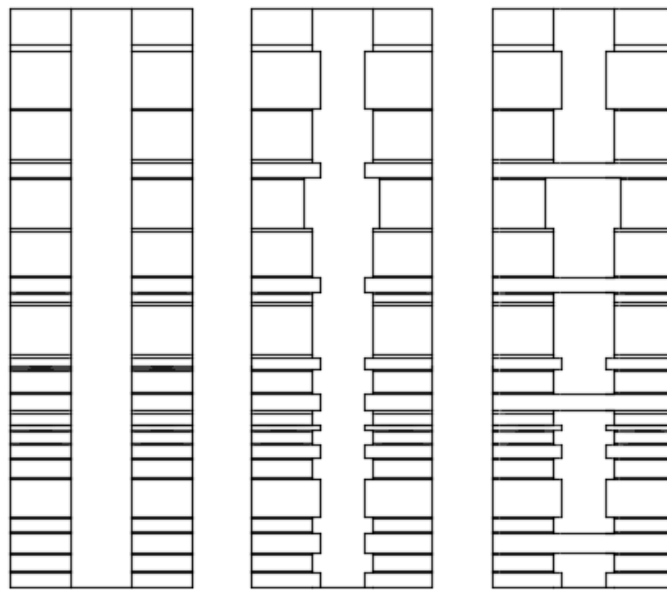


Fig. 4.16 - Drawings of Sectional Wall 2, Simple (Left); Interlocking (Central); Connected (Right)

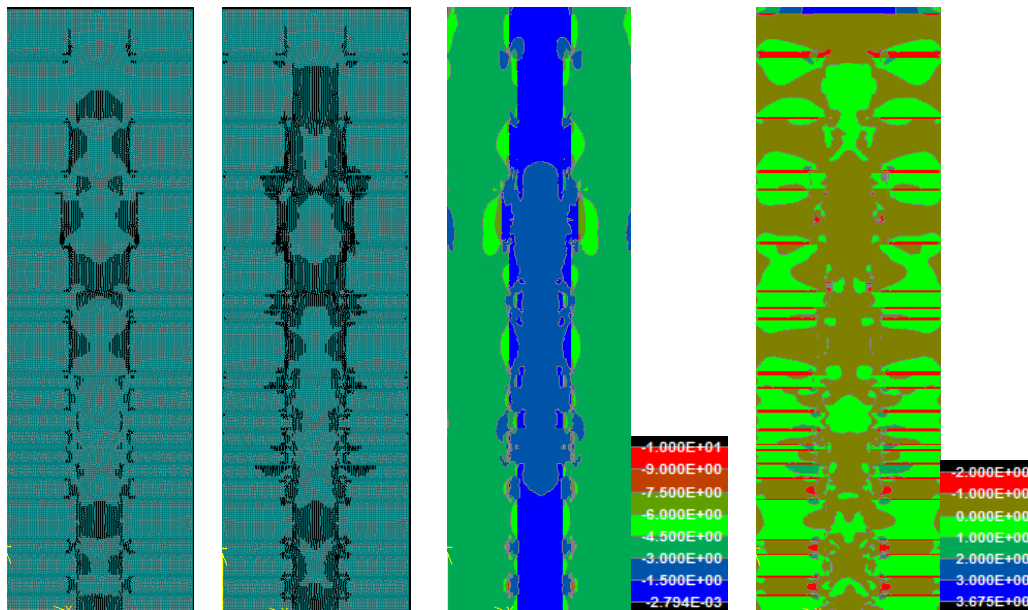


Fig. 4.17 - Cracking pattern of Interlocking Model 2 (5<sup>th</sup> and last steps) , Vertical  $\sigma_z$  and Horizontal  $\sigma_x$  stress distribution

As the previous results, also in this case it is noticeable a progressive evolution of cracks within the rubble and a distribution of vertical stress similar to the first sectional wall. In this case the peak compressive stress increased by 10% more the previous results, with a slight decrement of elastic modulus.

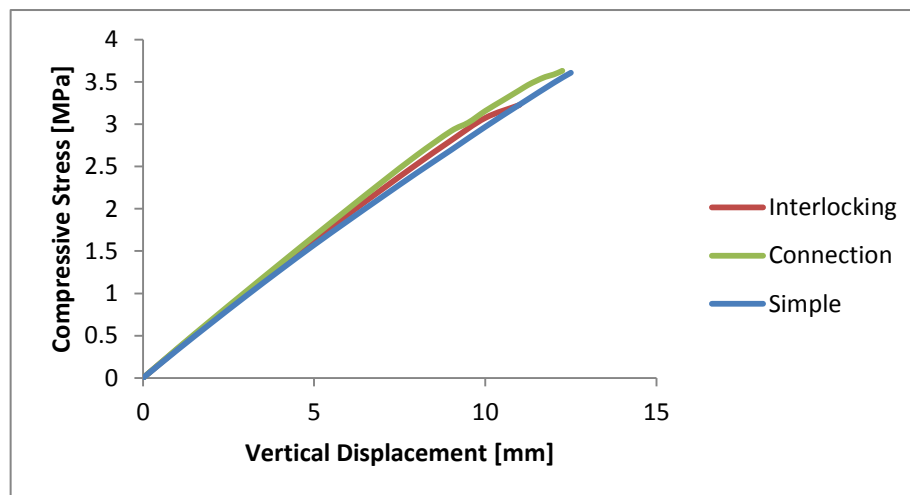


Fig. 4.18 - Comparison curves Displacement – Vertical Stress of the Sectional Walls 2

Tab. 4.5 - Results coming from the compressive tests

	Compressive Stress (MPa)	Elastic Modulus (GPa)
<b>Sectional Wall Simple 2</b>	3,60	1,26
<b>Sectional Wall Interlocking 2</b>	3,23	1,30
<b>Sectional Wall Connection 2</b>	3,63	1,32

#### 4.5. CURRENT LOADING SITUATION

Once defined the compressive strength, it is necessary to make a comparison with the currently stress to which the walls are subjected.

##### Dead Load Masonry

For the self-weight of the walls a value of the density equal to  $20 \text{ kN/m}^3$  was considered. Taking into account the portion of walls with thickness 1,2 m and an height of 16,5 m, the linear load is equal to:

$$G_{k1} = 400 \text{ kN/m}$$

##### Dead Load Timber roof

The total value consists in the weight of the truss of the main structure, of the beams supporting the platform, the beams supporting the ceiling and the weight of the roof cover. The last two are taken from the past thesis and are equal to:

$$G_{k2\_Ceiling} = 0,5 \text{ kN/m}^2$$

$$G_{k2\_Cover} = 0,93 \text{ kN/m}^2$$



For the self-weight of timber truss was necessary to take as reference the dimensions explained in the thesis of Giulia Facelli.

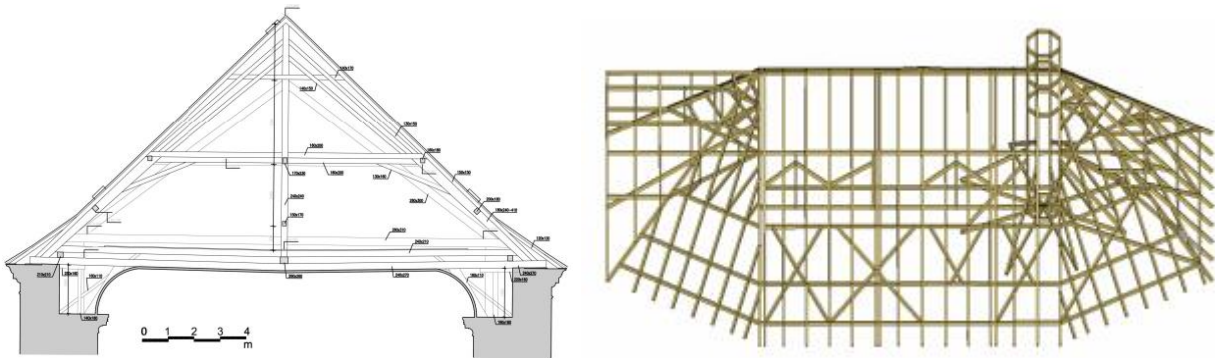


Fig. 4.19 - Details of the timber roof in St. Jacob (Facelli,2014)

Where:

$$G_{k2\_Truss} = 1,45 \text{ kN/m}^2$$

$$G_{k2\_platform+Beams} = 1 \text{ kN/m}^2$$

$$G_{k2\_Beams} = 0,5 \text{ kN/m}^2$$

The linear load distributed on the walls results equal to:

$$G_{k2} = 33 \text{ kN/m}$$

#### Live Load

According to EN1 the live load for maintenance works for category roofs H is equal to 0,4 kN/m<sup>2</sup>. However, according to the Table A.1.1. of EN0, along with snow, the combination factor is taken equal to zero.

#### Snow Load

The location of the church is an free plane zone, isolated and under 1000 m to the sea level.

From the EN1 the value of snow load is equal to:

$$S = \mu_i * C_e * C_t * S_k$$

Where,  $C_e$  and  $C_t$ , respectively exposure coefficient and thermal coefficient, could be considered equal to 1. The value of characteristic value of snow load for Broumov Region (Czech Annexes)  $s_k$  is equal to 2,25 kN/m<sup>2</sup>.

The value of snow load shape coefficient  $\mu$ :

$$\mu = \frac{0.8 * (60 - 44)}{30} = 0.427$$

The total value is equal to:

$$s = 0,96 \text{ kN/m}^2$$



Once defined all the loads, in order to find the maximum stress at the bottom of the walls the fundamental combination defined in EN0 was used, applying a factor of 1,35 to dead loads and a factor of 1,5 to snow load.

$$q = 1.35 * G_{k1} + 1.35 * G_{k2} + 1.5 * Q_{snow} = 625 \text{ kN/m}$$

The stress at the base results:

$$\sigma_{bottom} = 0.52 \text{ MPa}$$

As expected, the value of the maximum stress is much lower than the strength obtained by means of micro modelling. This is due to the decision of the builders to not realize masonry vault for the covering. In this way the weight coming from the roof is negligible in comparison with the self-weight of the walls.

In conclusion, the walls do not show defect in the bearing capacity and the damages detected has to be correlated with other causes.

#### 4.6. DISCUSSION RESULTS

If the pictures showing the vertical stress distribution are interesting in order to detect how the load spread among different materials, the study of the horizontal stress is also important to study the interaction between the mortar and the stones. In fact, when subjected to pure compression, the mortar tends to expand but it results confined by the units, determining a state of tangential interaction and causing a state of triaxial compression in the mortar and a state of biaxial tension in the units, as shown in Fig. 4.20. In the picture is also reintroduced a particular of the first sectional wall investigated, that matches well with the previous statement, where in red is the mortar subjected to compression.

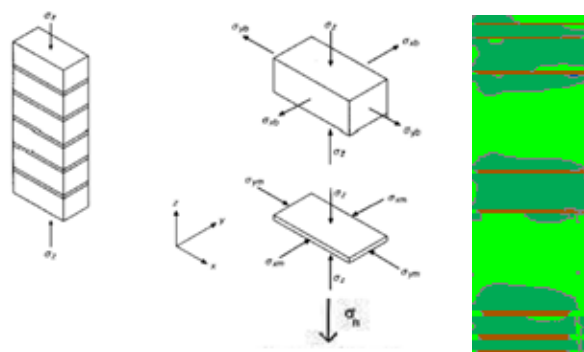


Fig. 4.20 - Typical stress state of a prism of masonry (Como,2010); Details of  $\sigma_z$  in the longitudinal wall 1

The results obtained after the micro modelling analysis of longitudinal and transversal section of walls, show a range of 2,9-3,6 MPa for compressive strength and 1-1,4 GPa for the elastic modulus. These values can be validated making a comparison with reliable standards, as Italian code. Italian code put a lot of efforts in order to determine mechanical parameters of different typologies of walls, express in the table C8A.2.1 reported in the next picture (Fig. 4.21).

The two typologies most interesting, for the comparison, are the first two groups about irregular stone masonry (with erratic and irregular stones) and masonry with infill core, where the upper values reach 3 MPa for compressive strength  $f_m$  and 1,44 GPa for elastic modulus E.

In conclusion the obtained results seem reasonable and reliable to be applied in the further 3D modelling.

Tipologia di muratura	$f_m$ (N/cm <sup>2</sup> )	$\tau_0$ (N/cm <sup>2</sup> )	E (N/mm <sup>2</sup> )	G (N/mm <sup>2</sup> )	w (kN/m <sup>3</sup> )
	Min-max	min-max	min-max	min-max	
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	100	2,0	690	230	19
	180	3,2	1050	350	
Muratura a conci sbozzati, con paramento di limitato spessore e nucleo interno	200	3,5	1020	340	20
	300	5,1	1440	480	
Muratura in pietre a spacco con buona tessitura	260	5,6	1500	500	21
	380	7,4	1980	660	
Muratura a conci di pietra tenera (tufo, calcarenite, ecc.)	140	2,8	900	300	16
	240	4,2	1260	420	
Muratura a blocchi lapidei squadriati	600	9,0	2400	780	22
	800	12,0	3200	940	

Fig. 4.21 – Extract from the Table C8A.2.1 of Circolare 2009 (Italian Code)

## 5. SOIL INTERACTION

### 5.1. OVERVIEW INTERACTION MODELS

As previously mentioned, the poor condition of the soil is a source of concern for the future stability of the church. For this reason, it is necessary to make an effort in order to get reliable results about the interaction between the soil and the church. This kind of analysis in professional life is unlikely handled, preferring more simplified approaches. Usually the study of the structure is separated from the study of foundation, imposing fixed supports at the base. This approach could be considered enough accurate when the soil is much more stiff than the above structure. This hypothesis is not more respected when the structure is particularly heavy and rigid, as could be a masonry building, and the soil not too much stiff, as soft clay. Different efforts have been done in literature to define the best approach for this kind of analysis.

The first method, widely used still nowadays, is the Winkler method (Winkler, 1867, Hetenyi, 1946), that characterizes the subsoil with a linear relationship between the vertical displacement of one point and the pressure acting on the same point:

$$p = Kw$$

Where  $K$  is the “subsoil stiffness” that could be seen as the ratio between the load and the settlement. Being the settlement depending on the loads, soil properties, shape and dimension of foundations, also  $K$  is depending on these parameters, making hard a direct correlation only with the typology of soil. On the other hand, as the results in terms of internal forces are not very sensitive at the variation of  $K$  values, it is enough to determine a reasonable value of  $K$ . So, the Winkler method defines the subsoil as a set of vertical springs, acting independently each other, leading to a strong assumption that is not realistic (Fig. 5.1).

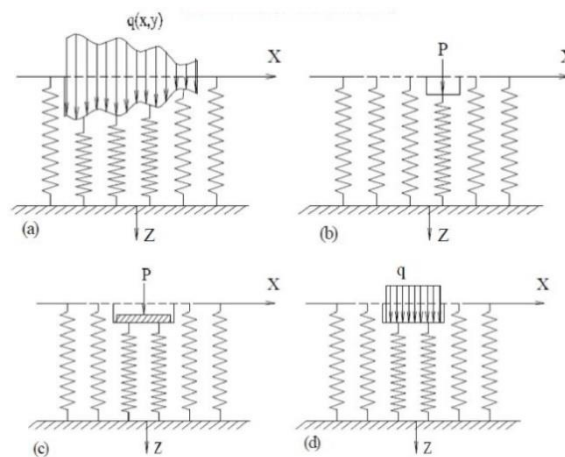


Fig. 5.1 – Winkler theory drawing (Chandra, 2014)

Several studies were carried out in order to define new models enable to capture aspects that Winkler approach could not get. It is possible to mention models elaborated by Grasshoff, by Hetenyi, by Vlasov and by Pasternak. All these models have as goal to incorporate the effect of the shear, leading

to the definition of two or more parameters, instead of only one as in Winkler formulation (Viggiani, 1993).

The Pasternak model takes into account a thin elastic membrane connecting the springs at the top, and subjecting them to a constant horizontal tension, simulating a more realistic behaviour (Fig. 5.2).

The commonly used expression for Pasternak is written as:

$$C_1 w - C_2 \Delta w = q$$

Where the constant  $C_1$  and  $C_2$  represent respectively compressive and shear deformability.

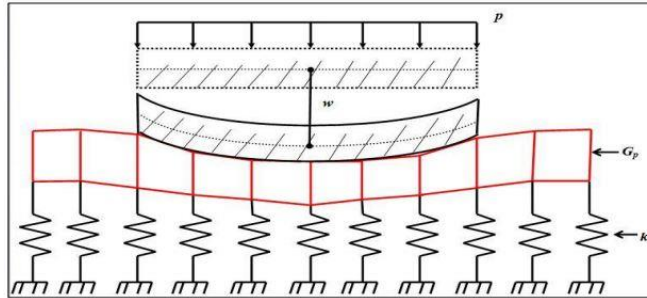


Fig. 5.2 Pasternak theory drawing (Breeveld)

A completely different approach is the continuum one, where the subsoil is modelled as a continuous distributed material. Analytical solution for continuous approach was elaborated by Boussinesq in 1885, who defined the subsoil as a semi-infinite, homogenous, isotropic, linear elastic media. The vertical stress is determined under hypothesis of vertical loads and infinite strip loads. In the formulation the self-weight of the soil is ignored, so the final result is directly linked to load applied.

$$\sigma_z = \frac{3Q}{2\pi z^2} \frac{1}{\left(1 + \left(\frac{r}{z}\right)^2\right)^{2.5}}$$

Where  $r$  is the horizontal distance from the point where the stress is computed, to the projection of the point where the load is applied, and  $z$  is the depth.

A different model is the one elaborated from Westergaard, that introduced an anisotropic material. Its formulation resulted to be very useful in case of alternation of layers much different each other. The Westergaard's expression neglects the horizontal movement assuming an almost infinite rigidity, due to the presence of water in soil (Viggiani, 1993).

## 5.2. DEPTH INFLUENCE ZONE

In the definition of the total deformation of the subsoil, it is fundamental to determine which portion of subsoil is actually subjected to the load of the structure in a relevant way. An easy and reliable approach is to consider the strain, in a certain point of the subsoil, as negligible when the stress, computed with Boussinesq or Westergaard, in that point is around 15-20% of the one applied on the surface.

However, in this thesis, as it will be explained, the depth of influence zone was determined by means of "Program Depth", based on Winkler Pasternak and the Theory of Structural Strength.

The program is based on the assumption that the deformability of virgin soil is considerable high, involving a minimal deformation during the subsequent unloading and reloading. This assumption leads to consider the soil as a incompressible layer until it experiences a stress higher than the previous one. Consequently, the depth of influence is defined by the point that reaches an effective vertical stress equal to  $\gamma h$  where  $h$  is the depth of excavation carried out (Kuklik, 2010). This is due to the concept that the original stress state is subjected to a alteration after the excavation, as occurs a variation in the highest stress level recorded in the loading history, that is often linked to the preconsolidation pressure (Kuklik, 2010). (Fig. 5.3)

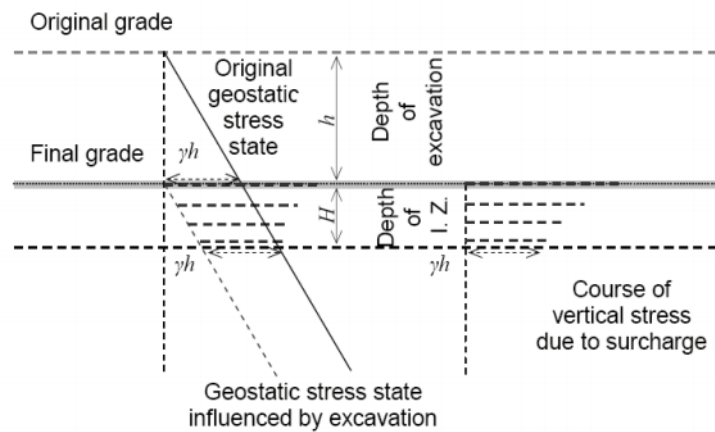


Fig. 5.3 (Kuklik, 2010)

The mathematic formula, used in “Depth Programme” is based on the deformation of an elastic subsoil layer, assuming, as in Westergaard, a null horizontal displacement, making stiffer the final response. The following expression comes from a uniform loaded strip of width  $2a$  (Kuklik,2010).

$$H = \frac{\pi a}{2} \left( \frac{2 - 2\nu}{1 - 2\nu} \right)^{\frac{1}{2}} \frac{1}{\ln \left( \sin \frac{\pi \gamma h}{2 f_z} + 1 \right) - \ln \left( \cos \frac{\pi \gamma h}{2 f_z} \right)}$$

From the formula is evident that the value of  $H$  is independent by the Young’s modulus, with the Poisson ration playing a significant role.

Similar to this approach is the Theory of Structural Strenght that is at the base of a further software used in this thesis, GEO5. In this method, a so named “ $m$ ” coefficient of structural strenght is defined. The influence zone is determined as the depth at which the increment in vertical stress ( $\sigma_z$ ) is used as a standard to equate with the original structural strength of soil ( $\sigma_{or}$ ) multiplied by the coefficient ( $m$ ) expressed as  $\sigma_z = m * \sigma_{or}$  and the settlement is expressed as  $s = f(\sigma_z, m, \sigma_{or})$ . The value of  $m$  is determined on the basis of the fundamental type of soil, consistency, consolidation and deformation modulus (Sejnoha, 2009).

### 5.3. SOIL DATA

The analysis of influence of the soil on the structure started from the definition of the basic characteristic of the soil under the church. For this purpose, two boreholes were drilled until a depth of almost 12 m, at the front and the back of the church. The several layers detected were grouped in five macro groups, having properties as the tables below.

Tab. 5.1 – Legend of the parameters of the soil

"Soil Type"	Estimated Soil Data
1	$E_{def} = 15.0 - 20.0$ [MPa], $\nu = 0.35$ [-], $\phi' = 20 - 25$ [°], $c' = 20 - 30$ [kPa], $\gamma = 19.5$ [kN.m <sup>-3</sup> ]
1+	$E_{def} = 20.0 - 25.0$ [MPa], $\nu = 0.35$ [-], $\phi' = 20 - 25$ [°], $c' = 20 - 30$ [kPa], $\gamma = 19.5$ [kN.m <sup>-3</sup> ]
2	$E_{def} = 20.0 - 25.0$ [MPa], $\nu = 0.35$ [-], $\phi' = 25 - 30$ [°], $c' = 25 - 30$ [kPa], $\gamma = 20.0$ [kN.m <sup>-3</sup> ]
3	$E_{def} = 25.0 - 30.0$ [MPa], $\nu = 0.35$ [-], $\phi' = 25 - 30$ [°], $c' = 30 - 40$ [kPa], $\gamma = 20.5$ [kN.m <sup>-3</sup> ]
3-	$E_{def} = 20.0 - 25.0$ [MPa], $\nu = 0.35$ [-], $\phi' = 25 - 30$ [°], $c' = 30 - 40$ [kPa], $\gamma = 20.5$ [kN.m <sup>-3</sup> ]

Tab. 5.2 – Subsoil layer distribution in the front of the Church

Distance from surface (m)	Description	"Soil Type"
0,00 - 1,50	Masonry Of Ventilation Channel (Fired Bricks) And Foundation Masonry, Masonry Partially Made From Sandstone	
1,50 - 1,55	Foundation Masonry - Compact Sample Almost Through The Whole Borehole Diameter, Light Coloured Ignimbrite (Paleoryolite?)	
1,55 - 1,80	Foundation Masonry, Ignimbrite And Light Rusty-Brown Sandstone Without The Presence Of Mortar	1
1,80 - 3,00	Silty Claystone With Numerous Sharp-Edged Siltstone Fragments (Max. Size 80 Mm, Ca 40 To 50 %), Fragmented Core, From Rusty-Brown To Light Ochroid Color	1
3,00 - 3,30	Smudged Core, Similar To Building Stone In The Frontage Masonry Of The Stone, Silty Claystone With Ochroid Color With Rusty-Brown Linings, Hard Consistency	2
3,30 - 4,50	Dtto	2
4,50 - 6,30	Dtto, Light Gray To Ochroid Color, In Some Places Rusty-Brown Color	2
7,40 - 7,55	Transition Of Rusty-Brown Color To Light Gray (Thanks To The Presence Of Clay Minerals And Absence Of Fe Pigments), Hard Consistency	2
7,55 - 7,70	Rusty-Brown Silty Claystone, Varied Layered, Layers Thickness 2 - 5 Mm, Hard Consistency	3
7,70 - 12,00	Compact Core, Silty Claystone, Hard Consistency, Consolidated, Brittle Breakage When Dividing, Rusty-Brown Color, In Some Places Gray	3

**Tab. 5.3 - Subsoil layer distribution at the back of the Church**

Distance from surface (m)	Description	"Soil Type"
0,00 - 0,04	Cement Dab Above The Ventilation Channel	
0,04 - 0,35	Brick Channel Masonry	
0,35 - 1,25	Foundation Masonry, Ignimbrite (Through Borehole Diameter, Thickness Ca 50 Mm)	
1,25 - 1,50	Dtto, Sharp-Edged Particles "Subbase", Without The Mortar Residues	
1,50 - 1,70	Rusty-Brown (In Some Places Greenish) Dusty Claystone, Hard, With Fragments, Compact	1
1,70 - 3,00	Dtto, The Borehole Core Disintegrated, Sporadic Stones Through Borehole Diamtere	1+
3,00 - 5,20	Dtto, Compact Rusty-Brown Core	2
5,20 - 12,50	Dtto - Hard Consistency	3
6,20 - 6,50	Disintegrated Core, Solid "Small Blocks" - Hard Consistency	3
6,50 - 7,20	Thin Lamination From Significantly Lighter Material, In Some Places Gray Fill	3
7,20 - 7,40	<b>Transitional Plastic Zone (Solid To Hard Consistency)</b>	3-
7,40 - 8,60	Compact Borehole Core, Hard Consistency	3
8,60 - 8,90	Dtto, Hard Consistency	3
8,90 - 9,00	Dtto, Hard Consistency, Brittle Core	3
9,00 - 11,00	Dtto, The Strength Increases With The Depth, Hard Core, Lighter Color, It Is Possible To Break Off By Hand On The Edges, Compact Fragments - Very Hard, Lower Consistency Limited In Some Places (See KP), Probably Thanks To The Drilling Influence	3
10,85 - 11,00	Rusty-Brown Dusty Claystone With Sharp-Edged Fragments (Probably Siltstones)	3
11,00 - 12,00	Dtto, Lightcoloured Striped Dusty Claystone, Compact, Hard Consistency	3

The foundations level is set at a depth of 1,5-1,8 m, respectively at the back and at the front of the church. If the depth of the first layer is similar in both locations, the difference in is evident in the total depth of the second main group that reach a depth of 7.55m in the front of the church, and a depth of 5,2 m in the back. A subsoil with hard consistency characterized the depth until a value of 12 m.

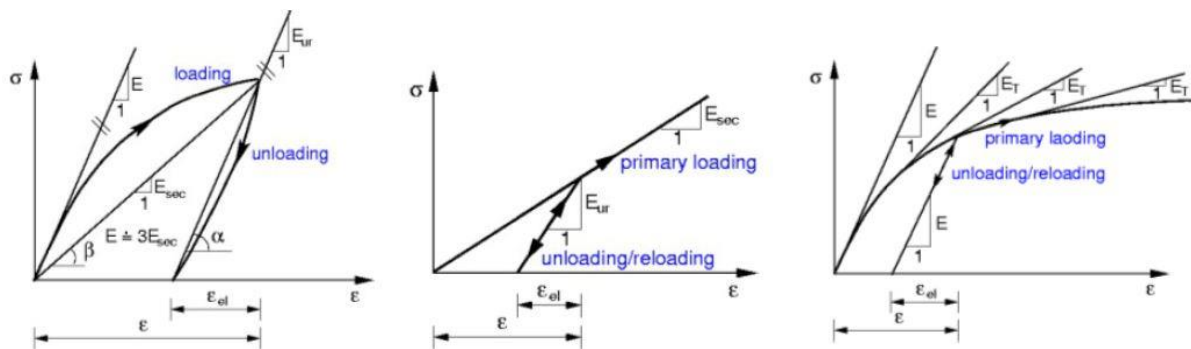
#### 5.4. GEO5 ANALYSIS

An analysis with the FEM software 2D "GEO5" was carried out in order to have sensitivity in terms of displacement that the church experienced and in terms of depth influence.

In GEO5 are available different constitutive models, basically divided into linear and non-linear models. The first category include Elastic and Modified Elastic, with the first one based only on Hooke's law, depending on the Elastic modulus and Poisson coefficient. The second one required the definition of the deformation modulus  $E_{sec}$  and unloading modulus  $E_{ur}$ .  $E_{sec}$ , as is noticeable from the real behavior of the soil (Fig. 5.4), is associated to the plastic yielding and decrease in stiffness, while

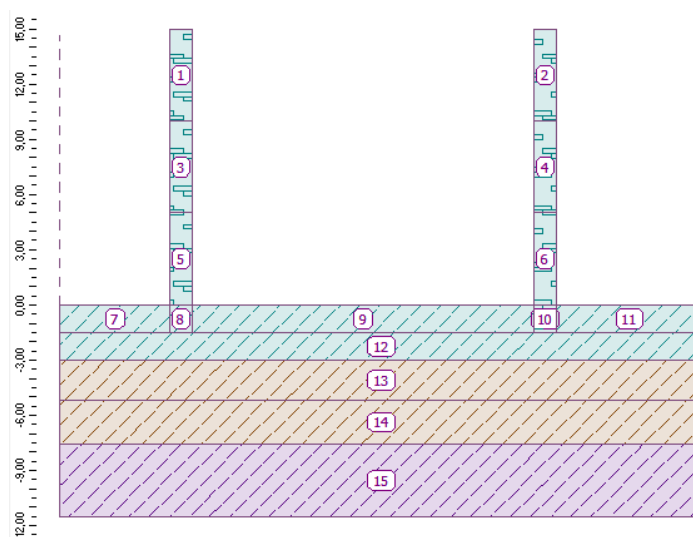
$E_{ur}$  characterizes how the soil behaves during unloading and reloading. A rough but reliable value is around three times the value of  $E_{sec}$ .

The nonlinear category is able to get better the real behaviour of the soils. By means of Mohr Coulomb Criteria, in GEO5 is possible to capture the change of stiffness of soil at each stress state. These criteria need the definition of more mechanical parameters, including the angle of internal friction ( $\phi$ ), cohesion( $c$ ) and dilatation angle ( $\psi$ ) (Sejnoha, 2009).



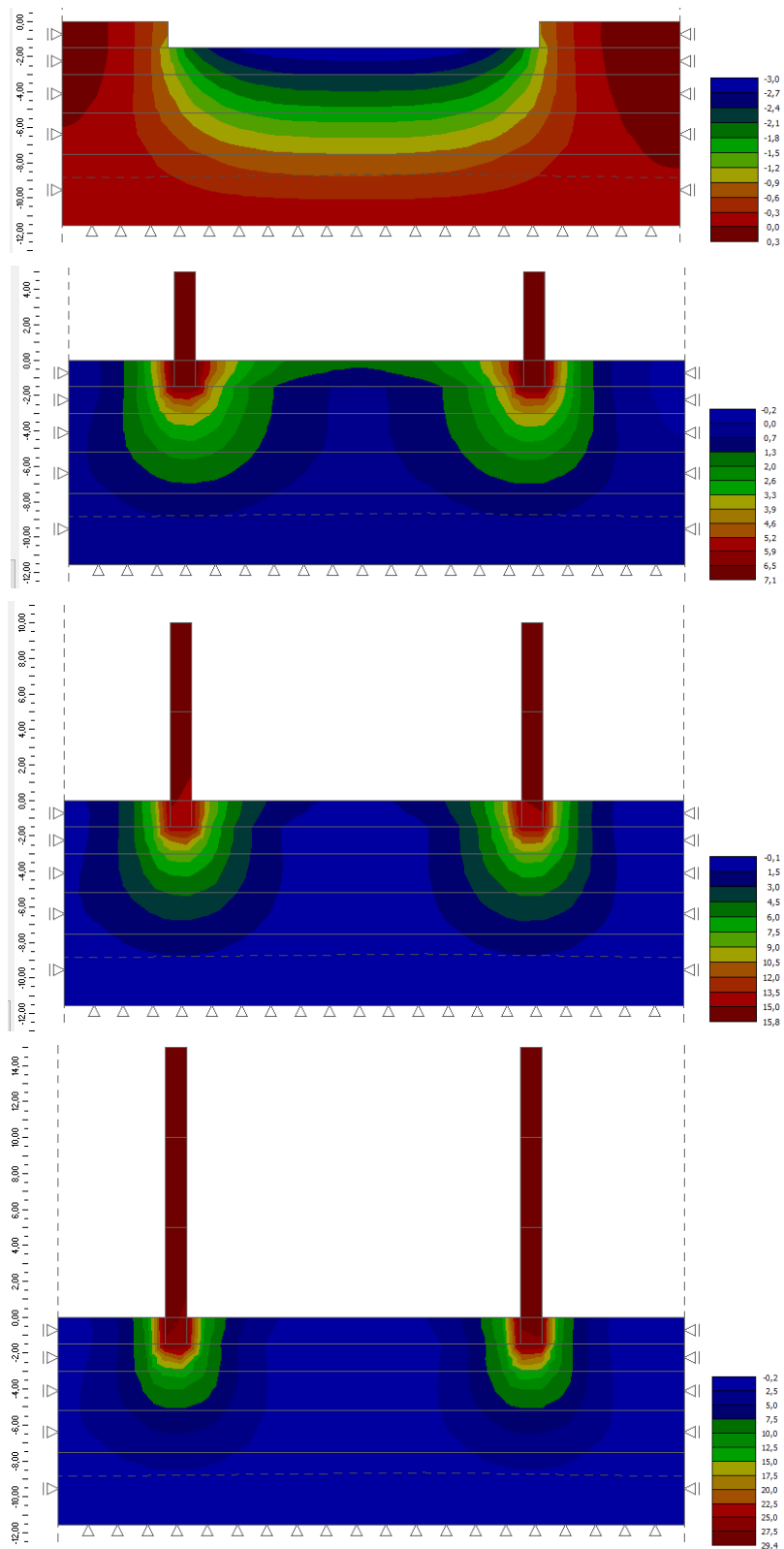
**Fig. 5.4 - Realistic behaviour of the soil (Left); Elastic model (Central); Mohr-Coulomb model (Right) (GEO5 Manual)**

The analysis carried out in GEO5 took into account only the transversal section of the walls along with the subsoil, as was necessary to respect the hypothesis of plan strain, on which GEO5 is based. The subsoil was modelled according to the results from the boreholes related to the front. The walls modelled are the ones in correspondence of the niches, with a thickness of 1,2 m. As in GEO5 is possible to reproduce the stages of construction and condition of soils, the load coming from the wall was divided in three different stages trying to simulate the real process of construction. In order to take into account also the weight coming from the roof, the density of the walls was increased accordingly from 20 kN/m<sup>3</sup> to 22 kN/m<sup>3</sup>.



**Fig. 5.5 – Modelling of the different layer of subsoil and transversal section of walls**





**Fig. 5.6 – Vertical displacements results**

The previous pictures express the evolution of vertical displacement during the process of construction reaching a maximum settlement between 25 and 27 mm and a depth influence of around 7 m.

## 5.5. CONVERSION OF SUBSOIL PROPERTIES INTO SPRING CONSTANTS

For the further 3D analysis it has been necessary a conversion of the properties of the subsoil in springs constant, as the one explained for Winkler and Pasternak methods. The first step is the calculation of Pasternak constants  $C1$  and  $C2$ , respectively compressive and shear deformability. Subsequently, from the relation between the total load  $f$  (kN/m) and the vertical displacement  $w$ , with  $b$  (m) the width of the foundation:

$$f = w * (2\sqrt{C1 * C2} + C1 * b)$$

It is possible to obtain the value of the “deformation value”  $k$ , being the ratio between the load applied and the settlement.

$$k = 2\sqrt{C1 * C2} + C1 * b$$

The  $C1$  and  $C2$  constants were determined by means of the already mentioned Program Depth. However, the programme allows the calculation only for one homogenous layer of subsoil. In presence of more layer, as in this case, it is necessary to carry out an homogenization of the entire subsoil, detecting an equivalent elastic modulus, coefficient of Poisson and self-weight. In order to consider the decreasing of the influence of the load on the deeper layers, a simple triangular attenuation of the load applied at the surface was applied. In this way, the deformations of each layer due to the corresponding load were computed, obtaining finally an overall deformation. From the overall deformation and the total depth considered, the strain was obtained and consequently the equivalent Elastic modulus.

Moreover, being the depth foundation unknown, it was necessary an iterative process. The first computation was done in relation of all depth explored by the boreholes. Afterward, once found the depth foundation and the involved layers of subsoil, a new equivalent Elastic modulus was computed.

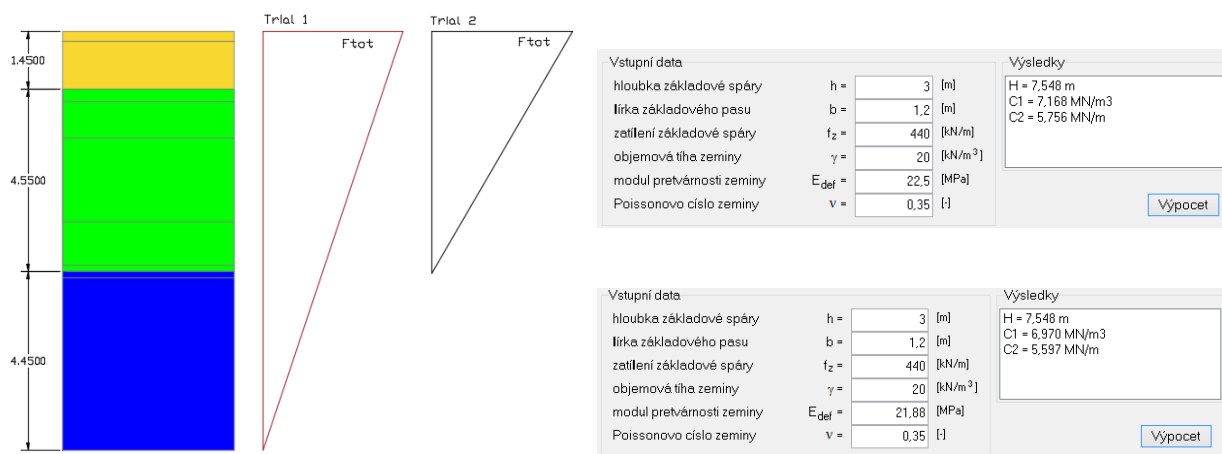


Fig. 5.7 – Procedure to find depth influence and Pasternak parameters for the soil in front of the Church

In the case of the borehole related to the front of the church, an initial value of 22,5 MPa was founded and a foundation depth equal to 7,55 m. After the second iteration, considering only the layers within this depth, the equivalent elastic modulus decreases slightly to 21,9 MPa, obtaining values for  $C1$  and

C2 as in table 5.4. The value of the depth of the foundation  $h$  was considered double than the actual one (1,5 m) in order to simulate the effect of pre-consolidation. The same procedure for Poisson's coefficient and density was not necessary as the negligible difference between the values.

Tab. 5.4 – Parameters of the soil in front

[Mpa]	[m]	[MN/m <sup>3</sup> ]	[MN/m]	[Mpa]
$E_{equiv}$	H	C1	C2	K
21,9	7,55	6,97	5,6	20,9

The same was done for the subsoil related to the backside of the church.

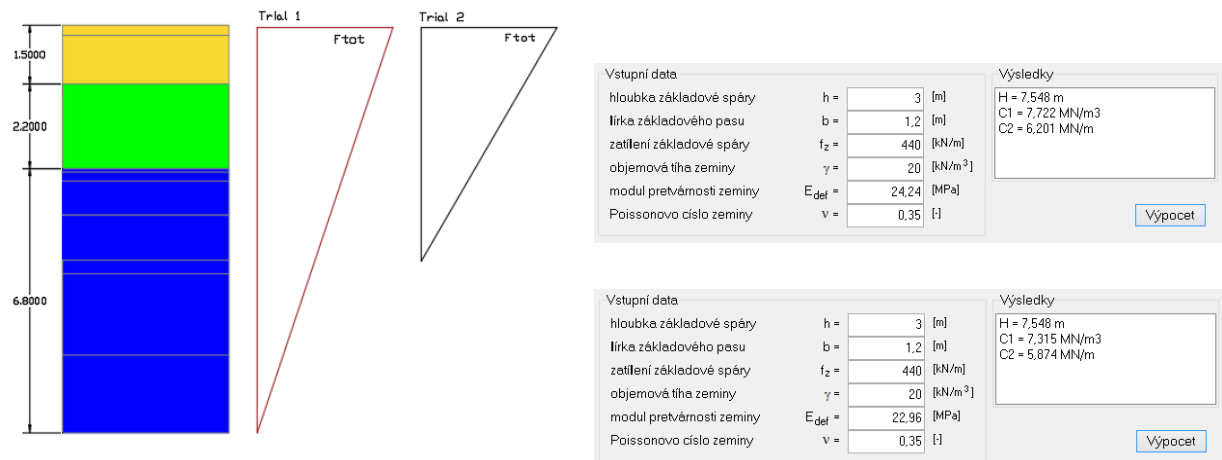


Fig. 5.8 - Procedure to find depth influence and Pasternak parameters for the soil at the back of the Church

Tab. 5.5 – Parameters of the soil at the back

[Mpa]	[m]	[MN/m <sup>3</sup> ]	[MN/m]	[Mpa]
$E_{equiv}$	H	C1	C2	K
22,9	7,50	7,31	5,87	21,9



## 6. 3D ANALYSIS

Once investigated the bearing capacity of the walls and the properties of the subsoil, it was possible to start a 3D analysis of the church, with the goal to explore its behaviour. The main purpose was the understanding of the cause of the actual cracks and the anticipation of possible future damages, creating a sort of catalogue of future scenarios, in order to make easier the future monitoring of the structure.

The 3D analysis was done by means of FEM software DIANA, enable to model the nonlinear behaviour of masonry and the cracks distribution.

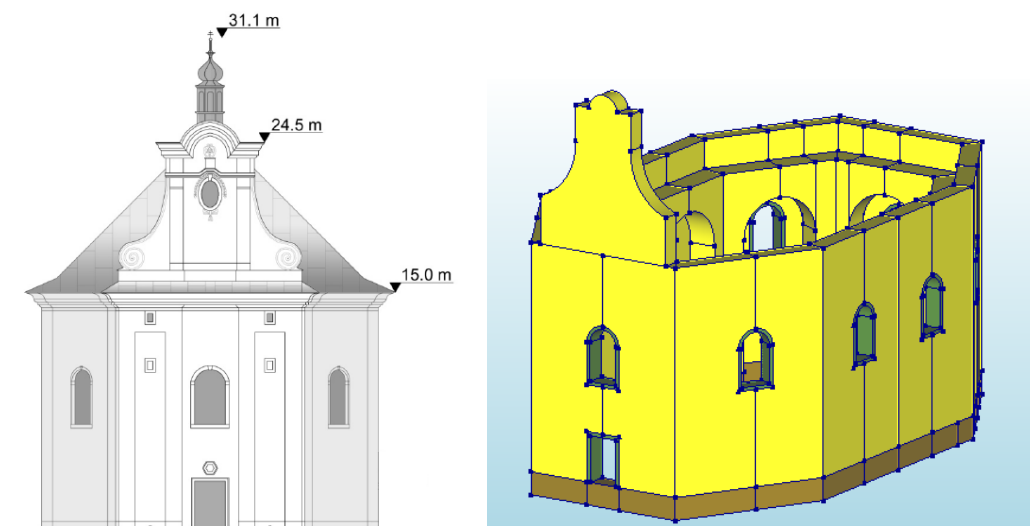
### 6.1. GEOMETRY AND MESH

The geometric model in DIANA was done after a slight simplification of the reality, taking away the small details that could have led to mesh issues, as the recesses of 1-2 cm in the walls (Fig. 6.1) or the particulars in the pillars and in the niches.

As the timber roof is not a structure that involves a thrust on the enclosure walls, as vault or a dome, it was not modelled and taken into account only as load.

The foundations (highlighted in brown in the pictures) have a depth of 1,5 m as the boreholes investigation revealed (Fig. 6.1).

As the purpose was the investigation of the global behaviour of the church and the evolution of possible settlements, it was neglected, in the model, the small sacristy beyond the church. Also, the huge difference in weight and in stiffness between the two structures could have involved numerical problems during the analysis



**Fig. 6.1 – Façade drawing of St. Jacob (Facelli, 2014); 3D Model in DIANA**

The library of DIANA allows the use of several elements for the modelling. In this case linear bricks (8 nodes) solid elements, called in DIANA "HX24L", were selected. The average size dimension is 0,4 m with a total number of nodes around 65000 (Fig. 6.2). This approach resulted to be a good compromise in order to obtain relatively fast analysis and reliable results. In fact, even if quadratic

elements (20 nodes) might allow more information and details in the results, the overall number of nodes would have exceeded 250000, making the analysis much more slower. As the final goal is the detection of the position of cracks and their evolution, the linear elements are more than enough for this purpose.

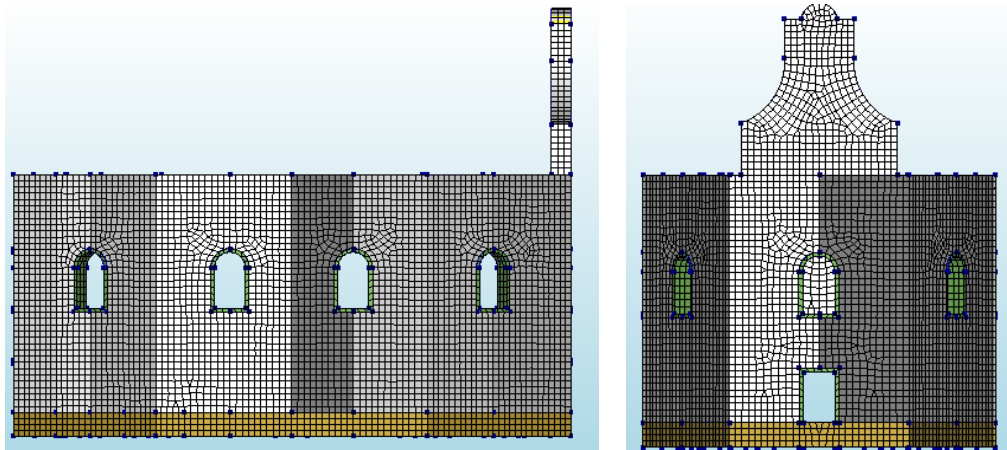


Fig. 6.2 – Mesh of the model in DIANA

## 6.2. MATERIAL PROPERTIES AND CONSTITUTIVE MODELS

The decision of which material properties used for the masonry is always a difficult task. In this case, the analysis carried out by means of micro modelling have contributed in reducing the several uncertain, acquiring more sensitivity in the definition of the parameters. In the chapter 4, different but close values of Young's modulus and compressive strength were detected. Starting from these the tensile strength was computed as 1/20 of the compressive strength, reaching a value closer to the one peculiar to the mortar.

DIANA offers different constitutive models for a wide number of analysis. For a cracking analysis, as in masonry, the “Total strain based cracked model” was selected (DIANA, 2017). For the compressive behaviour a parabolic function was selected:

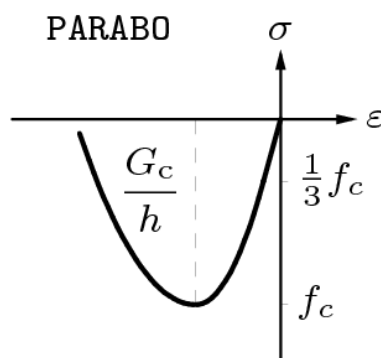


Fig. 6.3 – Parabolic constitutive model for compression

The definition of compressive fracture energy came from the Model Code 90 (Lourenço, 2008). The fracture energy could be related to compressive strength through the ductility parameter  $d$ , as:

$$G_{fc} = d * f_c$$

The value of  $d$ , for a compressive strength until 12 MPa, is taken equal to:

$$d = 2,8 \text{ mm} - 0,1 * f_c$$

For the tensile behavior, a softening tensile exponential curve was chosen:

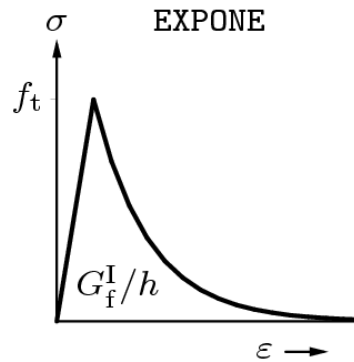


Fig. 6.4 – Tension exponential softening model

About the tension fracture energy, a value between 10 and 50 N/m is suggested. However, a value close to the minimum one could lead to serious issues of convergence during the analysis and it is more indicated in case of poor masonry.

In the following table are summarized all the values applied:

Tab. 6.1 – Linear and Non-Linear parameters used in the 3D Model

E (GPa)	$\nu$	$f_c$ (MPa)	$G_c$ (N/m)	$f_t$ (MPa)	$G_t$ (N/m)	$P$ (kN/m <sup>3</sup> )
1,3	0,2	3,0	7200	0,15	50,0	20,0

The lintels of the doors and of the windows were modelled as a linear material, in order to avoid unrealistic concentration of stress and early formations of cracks, that could lead to convergence issues since the first steps of the analysis.

At the bottom of the foundations, elastic springs were applied. From the “deformation modulus”  $k$  obtained in the previous chapter, the value of vertical stiffness  $K$  of the spring was obtained dividing for the thickness of the wall.

### 6.3. ELASTIC ANALYSIS

At first, a simple elastic analysis was carried out, in order to check banal mistakes in the mesh and in the material properties, and to make a comparison in terms of settlements with the result obtained with GEO5. As explained in chapter 5, the theories at the base of GEO5 and Programme Depth, with which the constants  $K$  were evaluated, are similar. So, comparable results in terms of settlements are expected. In the next picture, the result of vertical displacements is shown, with an average value of around 23 mm (Fig. 6.5) that is pretty close to the 27 mm computed with GEO5, validating in this way both the analysis.



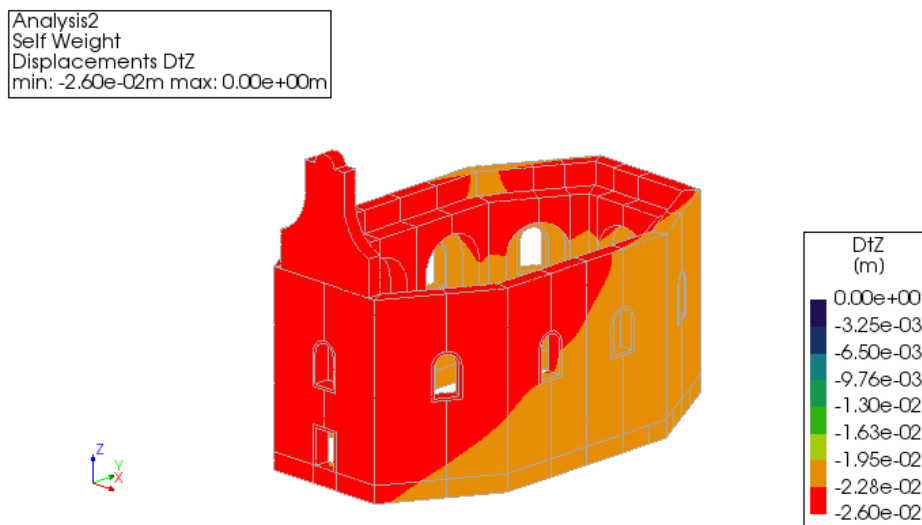


Fig. 6.5 – Vertical displacement results

#### 6.4. SETTLEMENT ANALYSIS (NON LINEAR ANALYSIS)

Once acquired confidence in the model, a set of nonlinear analysis were carried out with the purpose to identify the distribution of cracks as a result of differential settlements.

As was made clear since the introduction, the several uncertain that characterize this case study made necessary an analysis not focused on a fixed situation, but imposing different situations based on different hypothesis.

As mentioned in the damage overview, by a visual inspection some portion of the soil were detected as deteriorated. Basically, the locations correspond to the points where the not working gutters do not allow the raining water to run away, making worst the properties of the soil (Fig. 6.6). The most critical situation was detected in correspondence of the gutters coming from the roof of the sacristy (Fig. 6.6), creating concerns for the possibility of settlements in the back side of the church.

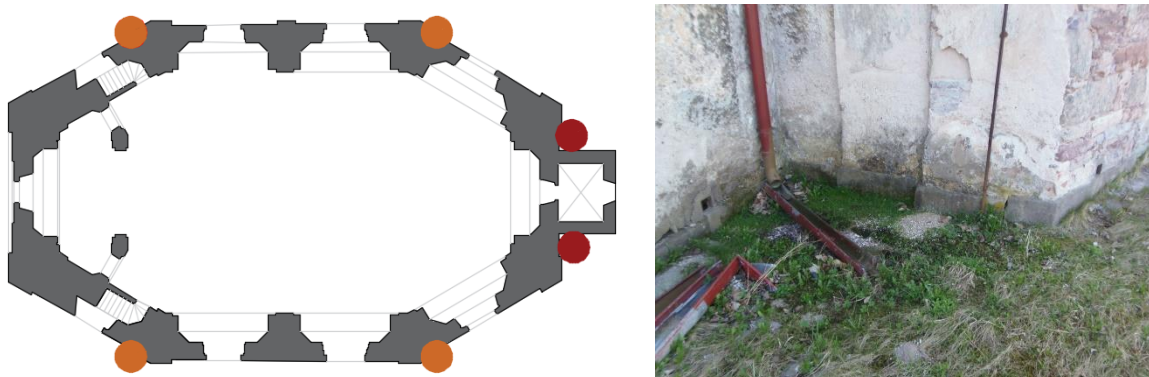
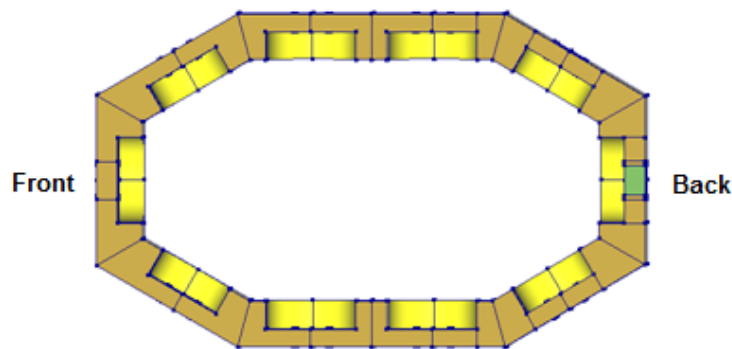


Fig. 6.6 – Localization of deteriorated soil (in red the worst condition); Detail of the worst situation

Obviously, being based on a simple visual inspection, it is not possible to be sure about the actual critical situation of the soil. In order to face with this difficulty, the analysis were carried out, changing the position of the soil supposed to be problematic.

However, after the application in the model of the self-weight, is not possible an evolution of the behaviour of the structure imposing just low properties in the stiffness of the springs. The effect of poor soil has been simulated imposing, after settled the self-weight, a progressive vertical displacement at the base of the foundation interested, determining a differential settlements.

In DIANA model, the foundation was divided in different zone, in order to make easier the application of displacements only in certain locations (Fig. 6.7).



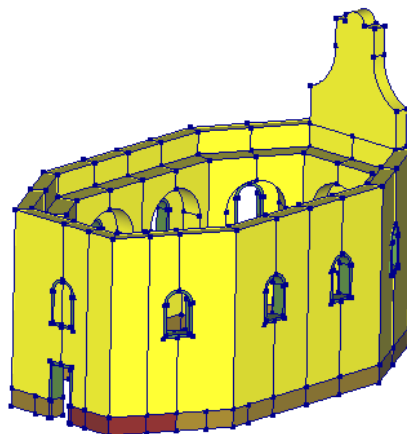
**Fig. 6.7 – Bottom point of view of the model**

Applying a progressive settlement divided in steps of 2,5 mm is possible to understand the first walls and locations involved in the cracking process, making possible at the end a final comparison with the actual situation. The angular distortion  $\beta$  consequent to the differential settlement is the main cause of damage.  $\beta$  could be seen, in this cases, as the ratio between the displacement and the pillars span. Probable structural damage is expected for value of  $\beta$  between 1/300 and 1/150 (Miranda & Ramos) that is the case of the analysis where a maximum displacement of 2,5 cm was applied.

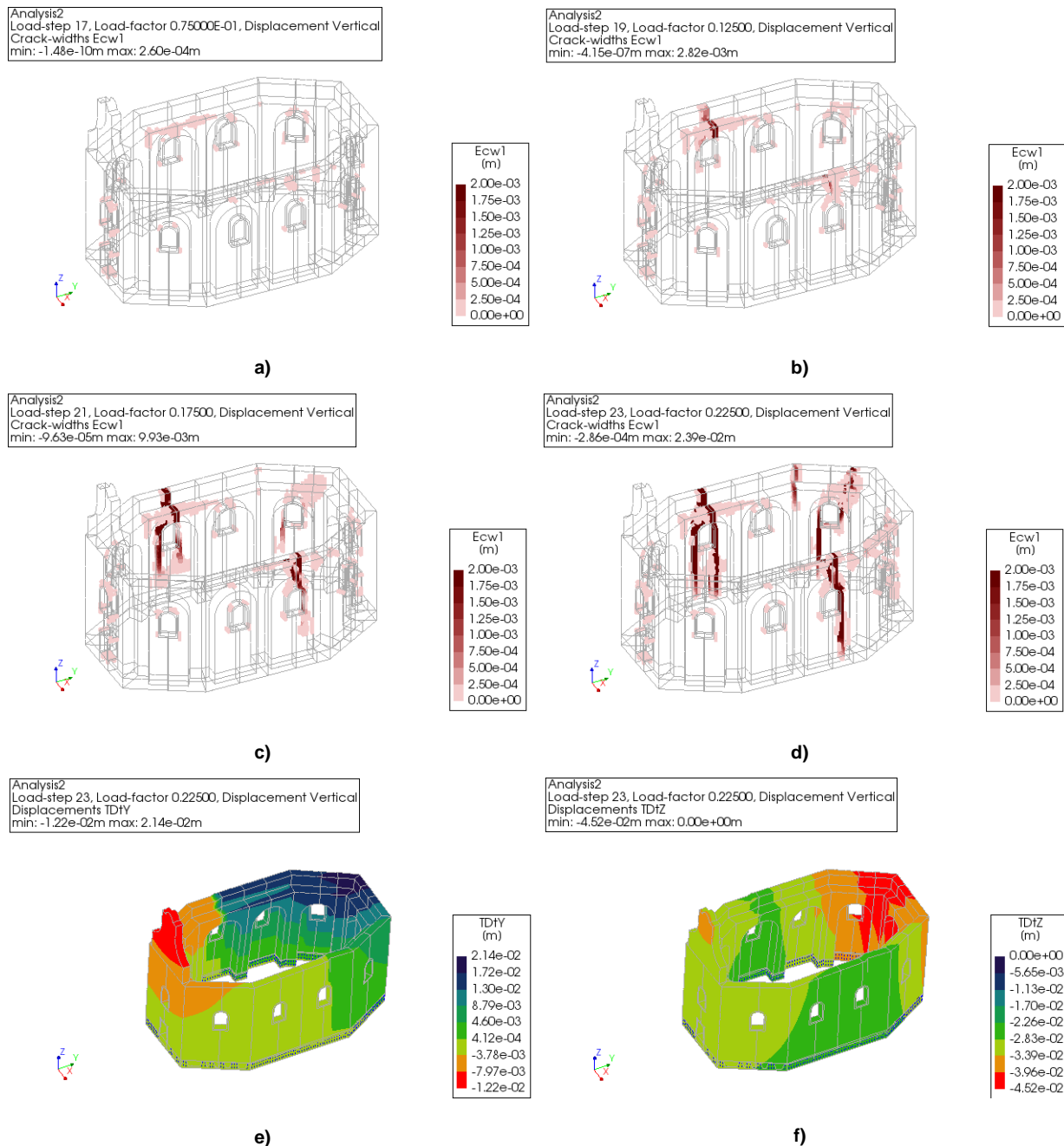
As said, different scenarios were explored, involving at first the soil at the back side, and finally, all the location source of concerns.

#### **6.4.1. 1° Scenario**

The first scenario involves settlements at the NW corner of the church, where actually the visual inspection detected the worst condition around the church, highlighted in red in the picture below.



**Fig. 6.8**



**Fig. 6.9 - Cracking pattern in different steps and horizontal-vertical displacement at the last step (1° Scenario)**

Taking as reference the following table (Burland,1977), it is noticeable that until a differential settlement of almost 1 cm, the effect on the walls in terms of damage is negligible, involving hairline cracks, above all upon the openings of the straight longitudinal walls. Even if, in these first steps the results do not look relevant, they have a huge role in the spirit of this thesis, as the goal is to arise the level of attention in the monitoring of the church. Anyway, at the increase of the displacement, it is possible to appreciate how the cracks spread, involving with moderate damages the niches in the longitudinal wall. Finally, also the “diagonal” wall is involved, especially at the corner of the opening and between the walls.

Degree of damage	Description of typical damage	Approximate crack width
(0) Negligible	Hairline cracks.	< 0.10 mm to 0.15 mm
(1) Very slight	Fine cracks which can easily be treated during normal conservation/decoration works.	~ 1 mm
(2) Slight	Cracks which can be easily filled and probably require re-decoration. Possible need of repointing to ensure weather-tightness.	< 5 mm
(3) Moderate	Moderate cracks which can be easily patched or masked by suitable linings.	5 mm to 15 mm
(4) Severe	Large cracks which require extensive repair work. Impair of functionality.	15 mm to 25 mm
(5) Very severe	Very large cracks which require major repair job. Danger of instability.	> 25 mm

Fig. 6.10 – Correlation between the width of the cracks and their level of danger (Burland, 1977)

#### 6.4.2. 2° Scenario

In the second scenario, all the back side of the church is involved, as highlighted in picture.

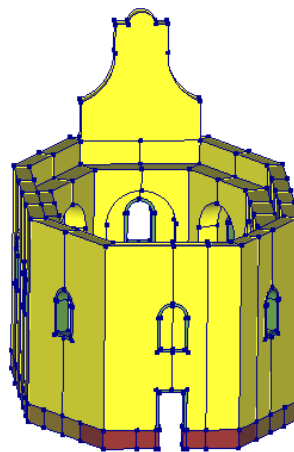
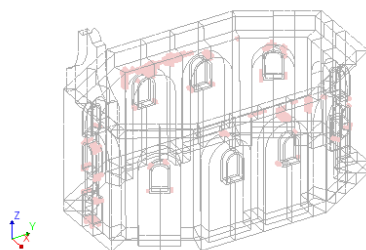


Fig. 6.11

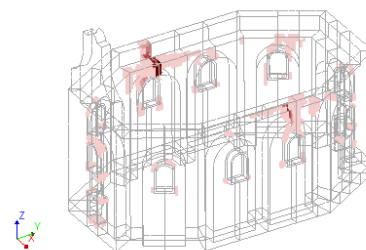
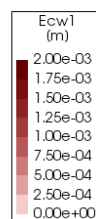
In this case, as expected, the distribution of damage is symmetric, but also in this case the first locations to be affected are the ones upon the openings of the longitudinal walls. In comparison with the previous scenario, here a severe damage was obtained with less imposed displacement, as the area involved is greater.

Analysis2  
Load-step 17, Load-factor 0.75000E-01, Displacement Vertical  
Crack-widths Ecw1  
min: -1.48e-10m max: 2.60e-04m

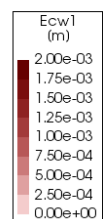
Analysis2  
Load-step 19, Load-factor 0.12500, Displacement Vertical  
Crack-widths Ecw1  
min: -4.15e-07m max: 2.82e-03m



a)



b)



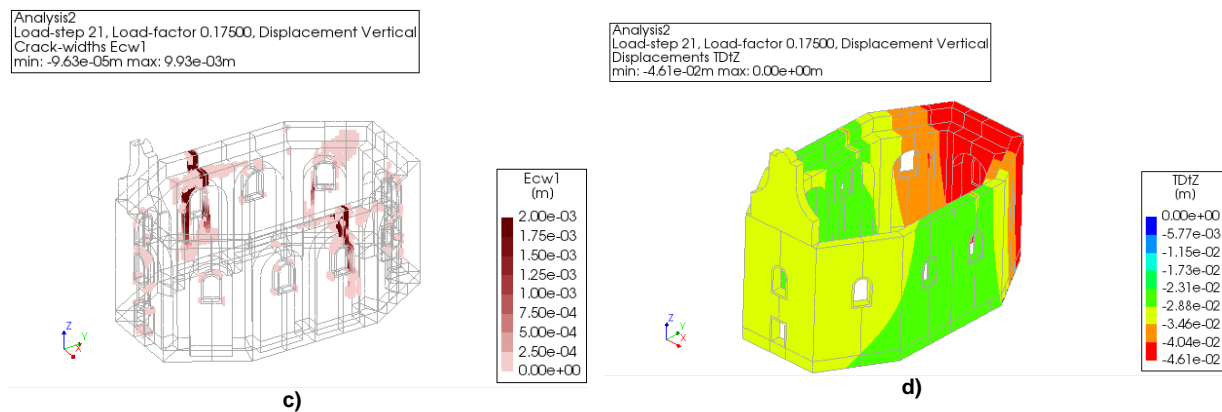


Fig. 6.12 - Cracking pattern in different steps and vertical displacement at the last step (2° Scenario)

### 6.4.3. 3° Scenario

In the third scenario, a new location was investigated. Along with the location in the NW corner, also the SW corner was subjected to displacement as the presence of a not well working gutter.

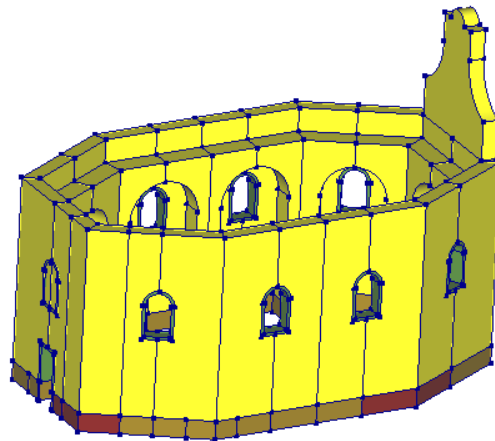
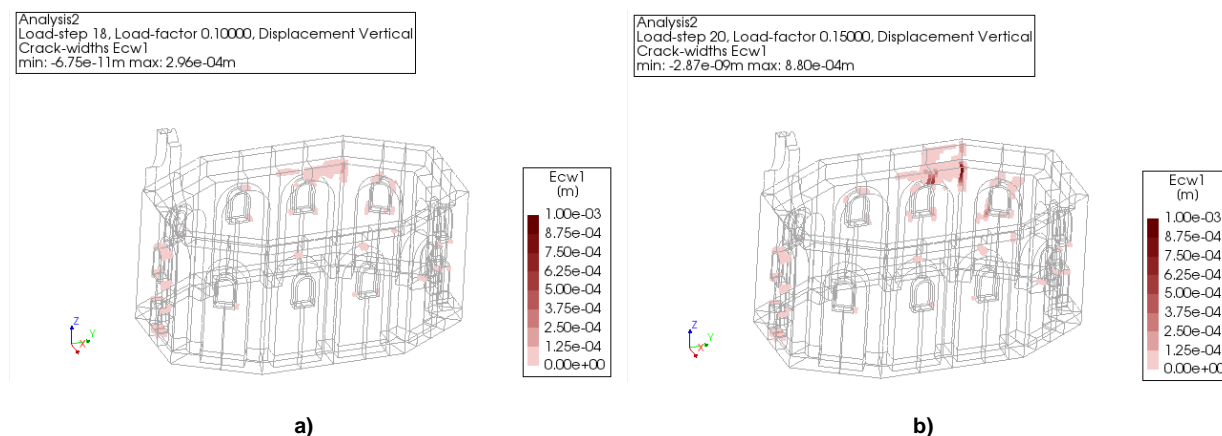
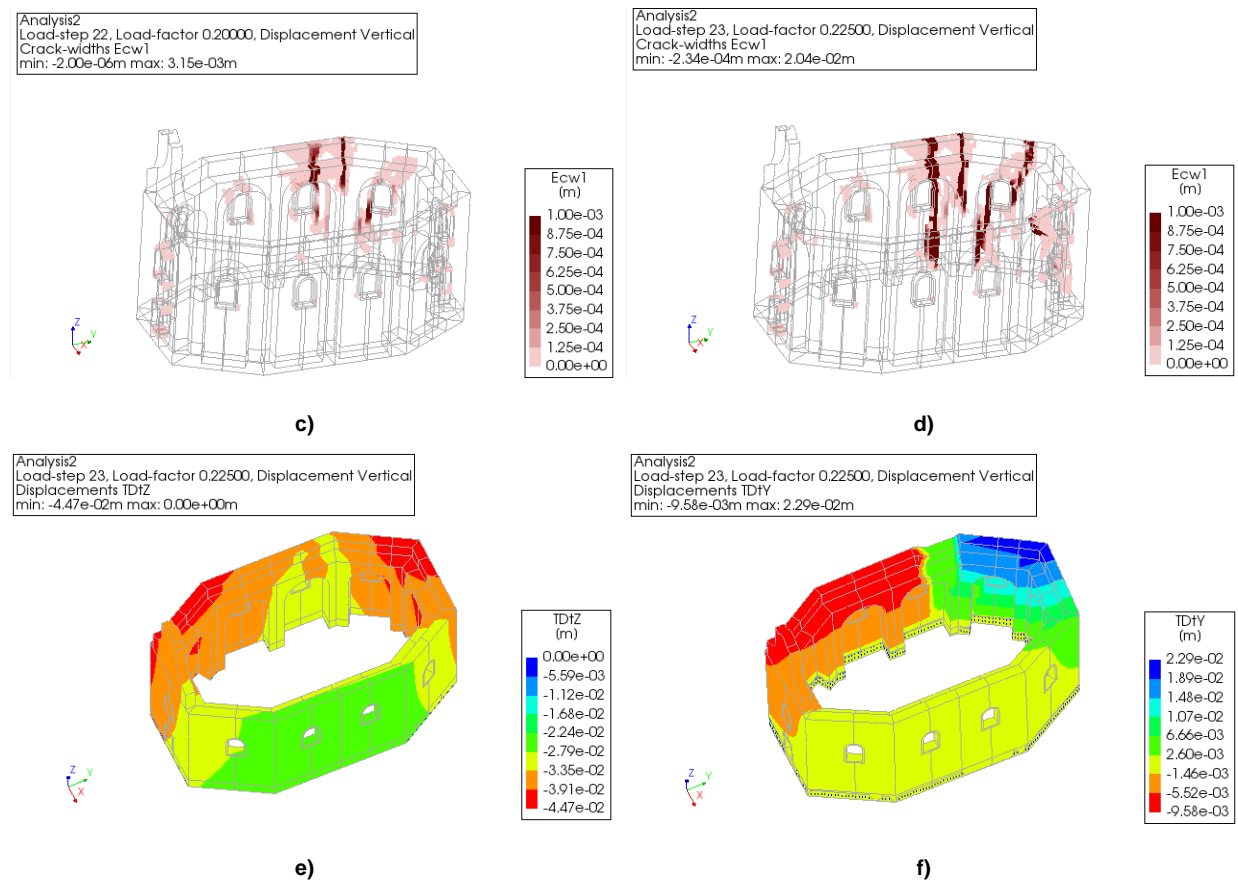


Fig. 6.13

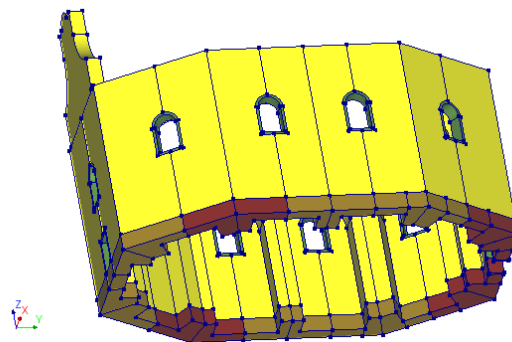




**Fig. 6.14 - Cracking pattern in different steps and horizontal-vertical displacement at the last step (3° Scenario)**

In this case, the damage occurs for first at the corner between the straight and diagonal walls, involving, afterwards, both walls. Also in this example, it is important to highlight that the initial cracks spread from the upper part of the walls, in the arches above the windows.

#### 6.4.4. 4° Scenario



**Fig. 6.15**

The last scenario proposed is the most extreme one, prefiguring settlements in all the locations interested by the presence of not good working gutters. The final result is a combination of the second and third scenario, involving finally also the main façade.



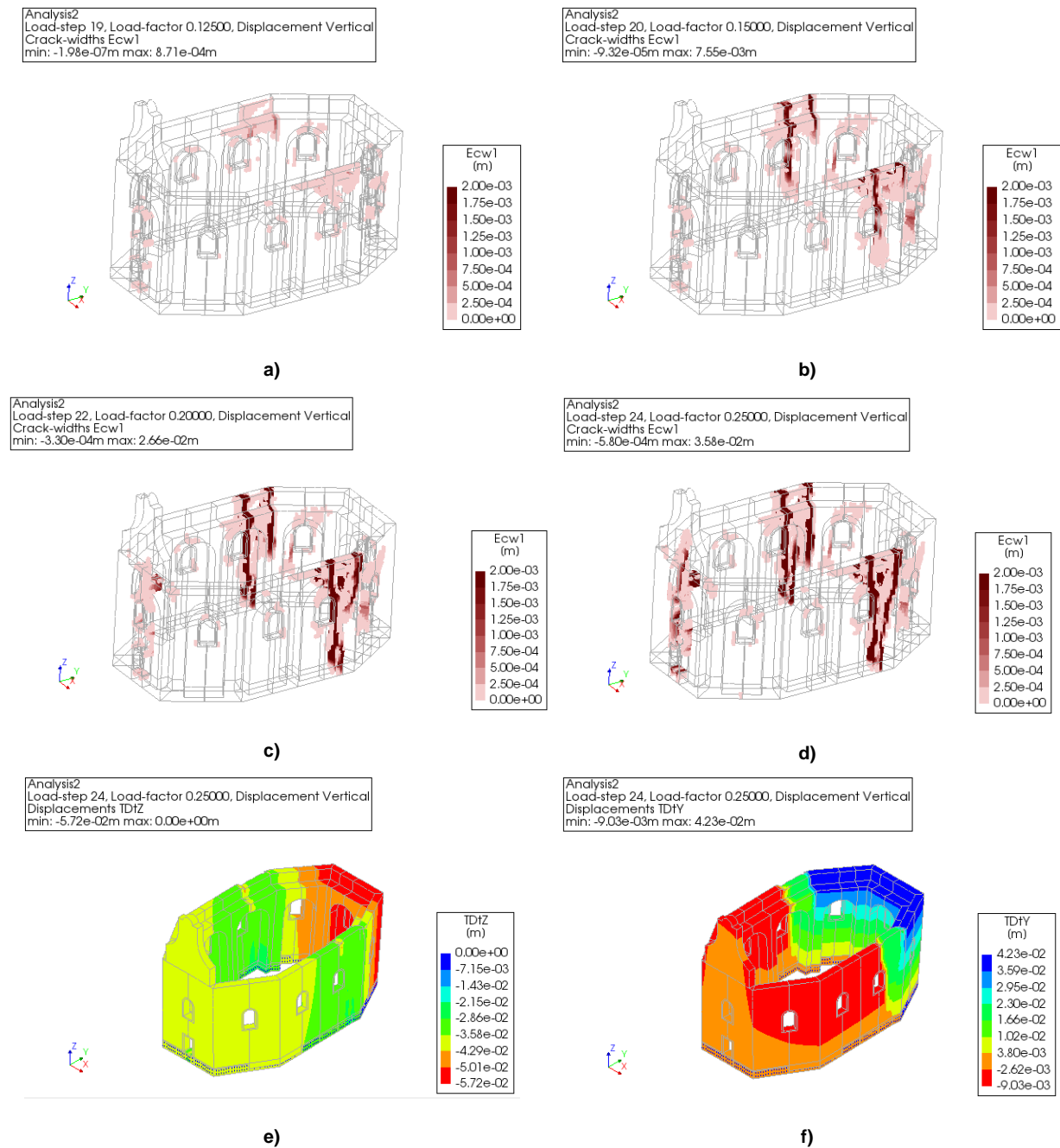


Fig. 6.16 - Cracking pattern in different steps and horizontal-vertical displacement at the last step (4° Scenario)

#### 6.4.5. Current cracking situation

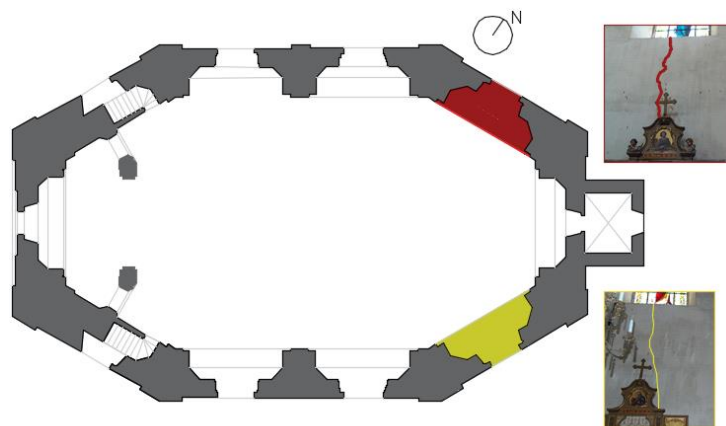
The results in the previous paragraphs give precious results in terms of most probable localization of cracks in case of settlement. However, the current cracks detected during the visual inspection match only partially with the ones obtained from the FEM model, making necessary the elaboration of different hypothesis. Some portions of the wall upon the openings show fine cracks (Fig. 6.17), that as observed during the first steps of the analysis is the early signs at the beginning of settlement. It is hard, anyway, at this stage make a definitive conclusion about a cause and effect.





**Fig. 6.17 – Cracking pattern above the opening in one of the niche**

As mentioned in the damage overview, the diagonal walls in the back side show vertical cracks under the openings. The crack at north is the most severe one, but the presence of a crack, even if thinner, in the opposite wall suggest a movement of the back walls (Fig. 6.18). However, as shown in the previous scenarios, differential vertical settlement involve at first cracks at the longitudinal straight walls, and only later cracks in the diagonal walls. For this particular kind of crack other hypothesis have to be elaborated.



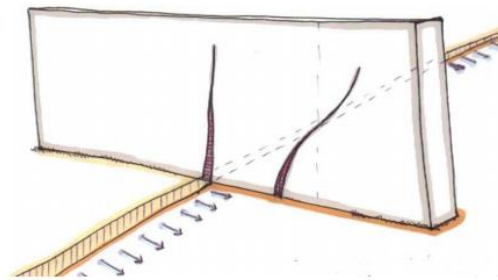
**Fig. 6.18 – Shape and locations of vertical cracks**

Horizontal cracks in the terminal part of a building could occur when the subsoil is highly sensitive to the presence of water. Especially soils that experienced variation of water table along with extreme changes in temperature, could lead to shrinkage phenomena involving the walls with vertical cracks. However, this case is more usual for building with not an excessive height (Di Francesco, 2008).

Also, changing in the level of water table could involve changing in the pressure below the wall and pillars, leading to vertical cracks. But, it would be necessary more span between the pillars and it would be difficulty to explain why the damage is localized only in one niche.

The vertical symmetric damage in two walls, unlikely involved during vertical displacements, could be related to horizontal settlement of the soil (Fig. 6.19). Even if the church is located on a slight hill, the slope is too low in order to create a movement. On the other hand, it is interesting what emerged from the archives about the works carried out in 1937 to build the drainage system around church. This

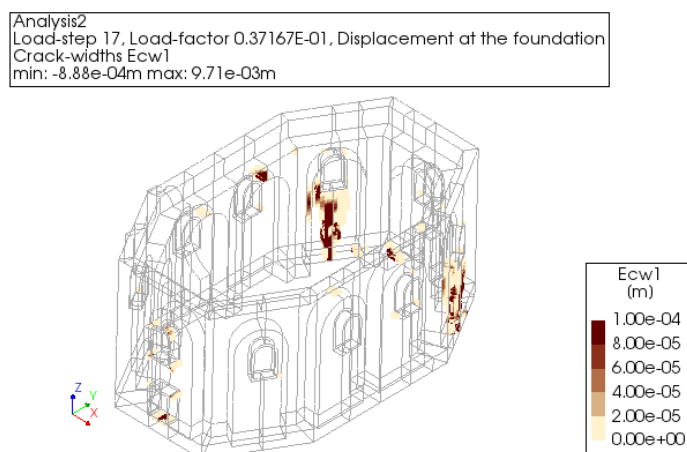
works involving excavation all around the church could have led to horizontal movement of the foundations in the back side.



**Fig. 6.19 – Typical crack coming from horizontal movement (Bianconi,2009)**

A DIANA model was run based on this hypothesis, similar in the concept to the previous one, but in this case imposing horizontal displacement at the bottom of back walls.

The results show a formation of horizontal cracks under the openings, as the reality, without involving other portion of walls. Even if the information available are not enough to confirm with absolute certainty this theory, it stands for an important starting point for future monitoring.



**Fig. 6.20 – Cracks Distribution linked to horizontal displacement at the base**

## **7. CONCLUSION AND RECOMMENDATIONS**

### **7.1. Conclusion**

The study done on the church of St. Jacob led to several considerations that will be here combined and synthesized as conclusion of this work.

The different approaches, used in the thesis, allowed to reduce the high level of uncertain, or at least to understand where it would be better concentrating efforts.

The first part of the thesis revealed how the church has been subjected to a process of deterioration under several point of view. The most severe problem is represented by the high level of moisture in the walls involving different damages, from biological colonization on the surface of the wall, to the deterioration of the stones.

However, the mainly contribution provided by the thesis is about the bearing capacity of the walls and the understanding of the behaviour of the structure under soil settlements.

From the micro modelling analysis, values of compressive strength, around 3.0 MPa, and Young's modulus, around 1.3 GPa, were obtained. These values, compared with reliable standards, are typical of a medium-good quality masonry. This allowed to acquire confidence in the skills of the ancient builders, and to overcome the concerns about the capacity of the wall compared to the load applied, six times less than the strength

The overall soil settlement were found by means of elastic analysis with GEO5 and DIANA, obtaining results around 2-3 cm of vertical displacement.

Afterwards, different scenarios of the church subjected to differential settlements were explored making clear the behaviour of the structure. In fact, it was clarified that vertical differential settlements are not the responsible of the current cracking pattern. However the analysis provided precious information for detecting future possible locations than might experience damages.

From the subsequent analysis carried out, it appears more probable a problem related to horizontal displacements, due to past works in site.

Based on the assumption done, from the structural point of view, the enclosure walls do not show serious problems enable to affect the stability. However, it is necessary to collect more information about possible differential settlements in progress. Once obtained, it will be possible a comparison with the scenarios here proposed and established the level of safety.

### **7.2. Future Recommendations**

The recommendations proposed here have the goal to suggest further tests and analysis, in order to reduce the number of uncertain and to continue the work started in this thesis.

It has to be taken into account that for the historical importance of the building, not invasive, compatible and durable interventions are suggested.

#### **Walls Features**

For first it is necessary to obtain more information about the geometry and the materials of the enclosure walls. At the moment, the properties of the infill is totally unknown, along with the one of the

mortar. An easy and cheap way to collect these information could be the use of an endoscopy camera. This required only the realization of a boreholes of a diameter of 2-3 cm, taking advantage of the locations where the thickness of the mortar is higher or in correspondence of softer stones. This test will help to realize if the filling is made of small size stones, rubble masonry or even poorer material.

The results obtained from the micro modeling could be compared with some tests on site. Double flat jack could provide both compressive strength and Young's modulus. However, in the peculiar arrangement of stones it would be difficult to identify two horizontal layers where to insert the flatjacks. So, it is suggested to use elastic wave methods as sonic tests, impact echo tests and tomography tests. Carrying out direct and indirect sonic tests, in different locations, would allow to obtain reliable results in terms of elastic modulus.

Along with the material properties, also the internal geometric features should be investigated. The endoscopy investigation could even help in determining the thickness of the leaves. However, impact echo tests would be the ideal in order to detect the internal shapes of stones and the eventual presence of interlocking. Moreover, as seen in the sectional wall analysis, the cracks in the internal leave occurred at the interface with the stones. Sonic tomography tests are able to give precious information about the presence of detachment in external leaves.

All these tests would allow to collect several data to be used in the models realized for this thesis, and also in further models.

### **Cracks and soil monitoring**

The vertical crack in the niches located at north has been related to horizontal movement of the soil, due to works carried out during the '40. However, to validate this theory, it would be useful to make a monitoring of an eventually extension of the crack. This could be checked applying a plaster mark on the crack. However, it could be easily broken by the effect of temperature changing that in Broumov region is high. So, it is suggested the application of a crack meter for a period of time that would allow to filter out the environmental effects. For this reason, the application also of sensors for temperature and humidity is necessary.

As mentioned in the conclusion, a monitoring of the soil settlements is a fundamental step for ensuring the future stability of the church. For economic reasons, these measurement will be not possible in several location, so at first, the best spot to be controlled should be the one at the back of the church, where the soil in correspondence of the sacristy seems more deteriorated. Along with periodic visual inspections, this could be enough to have under control the stability situation of the church.

### **General interventions**

As the concentration of waters around the church is source of concern, it is necessary to carry out an inspection in order to establish the effectively efficacy of the drainage system built in 1937, and eventually to plane a maintenance of that. At the same time, the gutters that do not work properly have to be fixed, avoiding the concentration of rainy water around the shallow foundation of the church.

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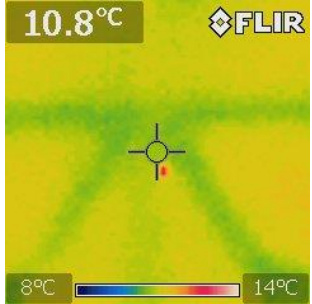
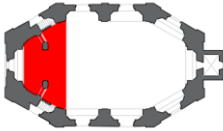
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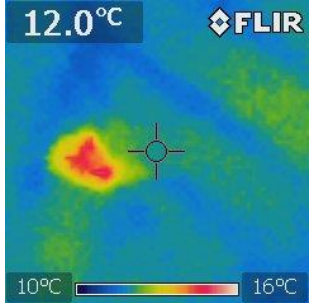
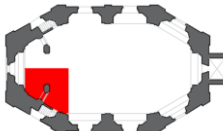
## APPENDIX A:

### LIST OF DAMAGES

Tab. A.0.1



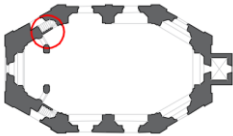
DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Cracks in the wood ceiling</b>	The ceiling presents several cracks due to the presence of the timber roof above, as the thermal picture shows.		
LOCATION			
			

Tab. A.0.2


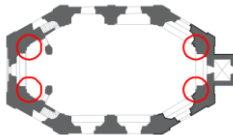
DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Hole in the wood ceiling</b>	Due to water infiltrations from the roof, a small portion of the ceiling is completely lost.		
LOCATION			
			




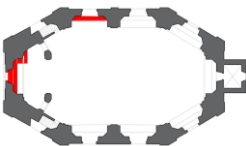
Tab. A.0.3

DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Cracks upon an opening</b>	A crack of almost 1 cm of thickness occurred in correspondence of corner the upper level, due to the presence of the opening.		
LOCATION			
			



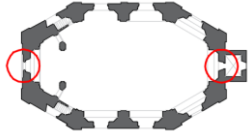
Tab. A.0.4

DAMAGE	DESCRIPTION	PHOTO
<b>Cracks at the corners</b>	At the corner of walls, cracks occurred due to the discontinuity of the shape.	
LOCATION		
		



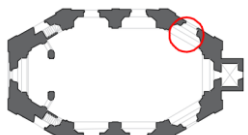
Tab. A.0.5

DAMAGE	DESCRIPTION	PHOTO
<b>Cracks in the plaster</b>	Above all in the walls located at east, some minimal cracks occurred. This could be related to concentration of salts under the surface.	
LOCATION		
		




Tab. A.0.6

DAMAGE	DESCRIPTION	PHOTO
<b>Cracks in the lintels</b>	The lintels experienced an high level of tensile stress, this led to vertical crack in the middle section.	
LOCATION		
		


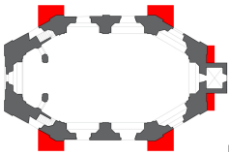
Tab. A.0.7

DAMAGE	DESCRIPTION	PHOTO
<b>Cracks upon and above the arch in the cell</b>	An horizontal crack characterizes the niche located at the north of the church.  The reasons for causing this damage could be various, and probably related to soil settlements.	
LOCATION		
		


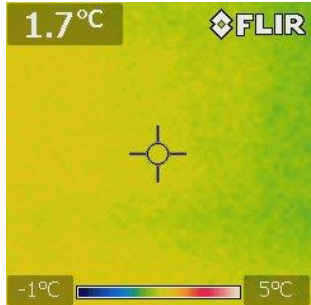
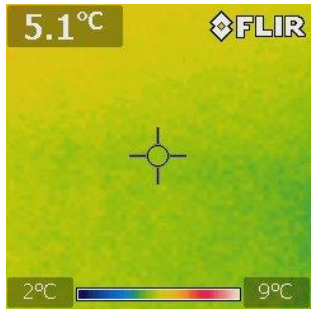

Tab. A.0.8

DAMAGE	DESCRIPTION	PHOTO
<b>Cracks in cross vault (Sacresty)</b>	<p>The cracks occurred upon the opening in correspondence of the cross vault may be marked as “compatibility cracking” due to the development of relieving arches or related to soil settlements.</p>	
LOCATION		
		


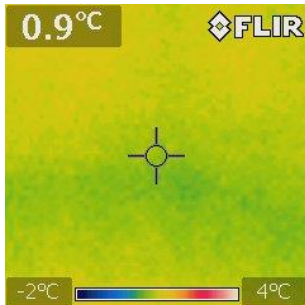
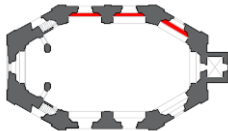

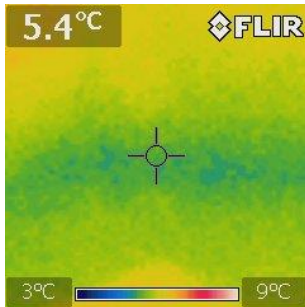
Tab. A.0.9

DAMAGE	DESCRIPTION	PHOTO
<b>Deterioration of soil in correspondence of gutters</b>	<p>The not working gutters do not allow the water to run away from the perimeter of the church, leading to soil deterioration.</p>	
LOCATION		
		



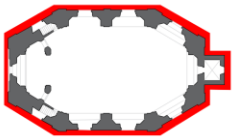
Tab. A.0.10

DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Rising dampness</b>	The walls at east and north-east experienced rising dampness, until a height of 2 m. This involved deterioration and loosing of the render.		
LOCATION			
			

Tab. A.0.11


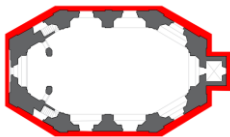
DAMAGE	DESCRIPTION	PHOTO	FURTHER INFO
<b>Rising dampness</b>  <b>Biological colonization</b>	The walls at north and north-west experienced rising dampness, until a height of 2 m, along with biological colonization.		
			

Tab. A.0.12

DAMAGE	PHOTO
<b>Loss Of Render</b>	 
LOCATION	
	
DESCRIPTION	
<p>The loss of the render affected all the surface of the external walls.</p> <p>The thinner one is basically completely gone, leaving the stones vulnerable to external actions.</p>	



Tab. A.0.13

DAMAGE	PHOTO
<b>Bulging of Render</b>	
LOCATION	
	
DESCRIPTION	
For reasons related to moisture and salt crystallization on stone surface, the render experienced phenomena of bulging, reaching in some locations values of several cm.	





## APPENDIX B

Fig. B.1 – Horizontal and vertical stress distribution in Longitudinal Wall 1 (Last step)

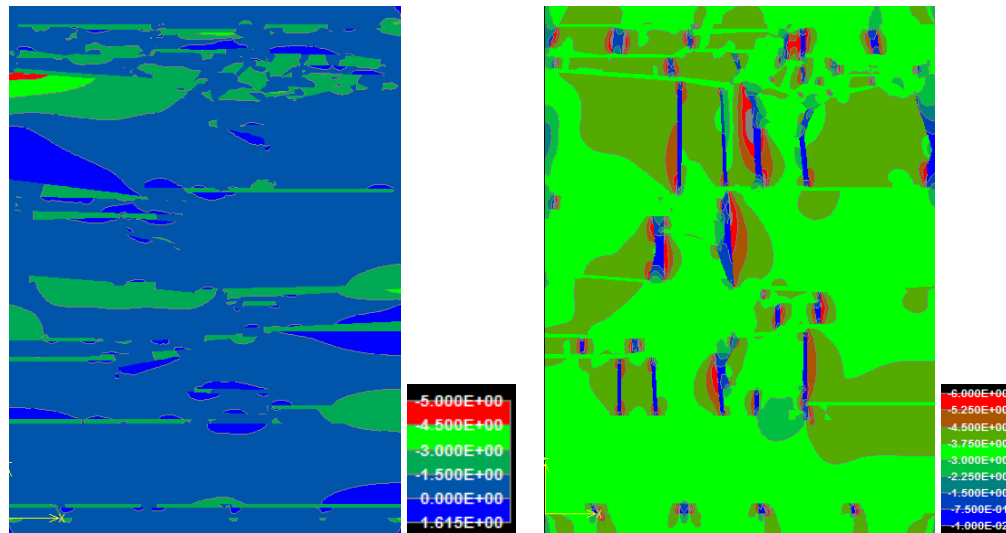


Fig. B.2 - Horizontal and vertical stress distribution in Longitudinal Wall 2 (Last step)

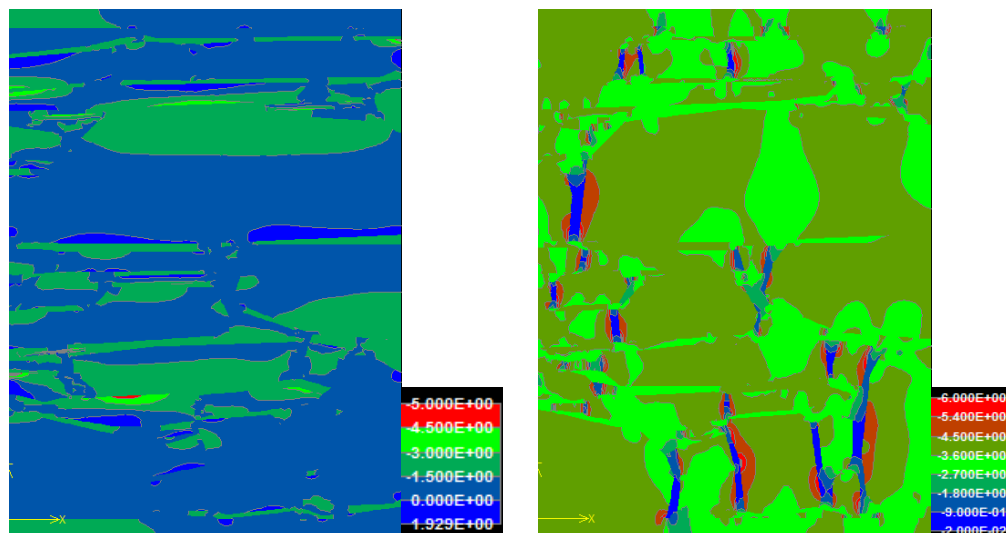


Fig. B.3 – Cracks pattern, vertical and horizontal stress distribution in Sectional Connection Wall 1 (Last Step)

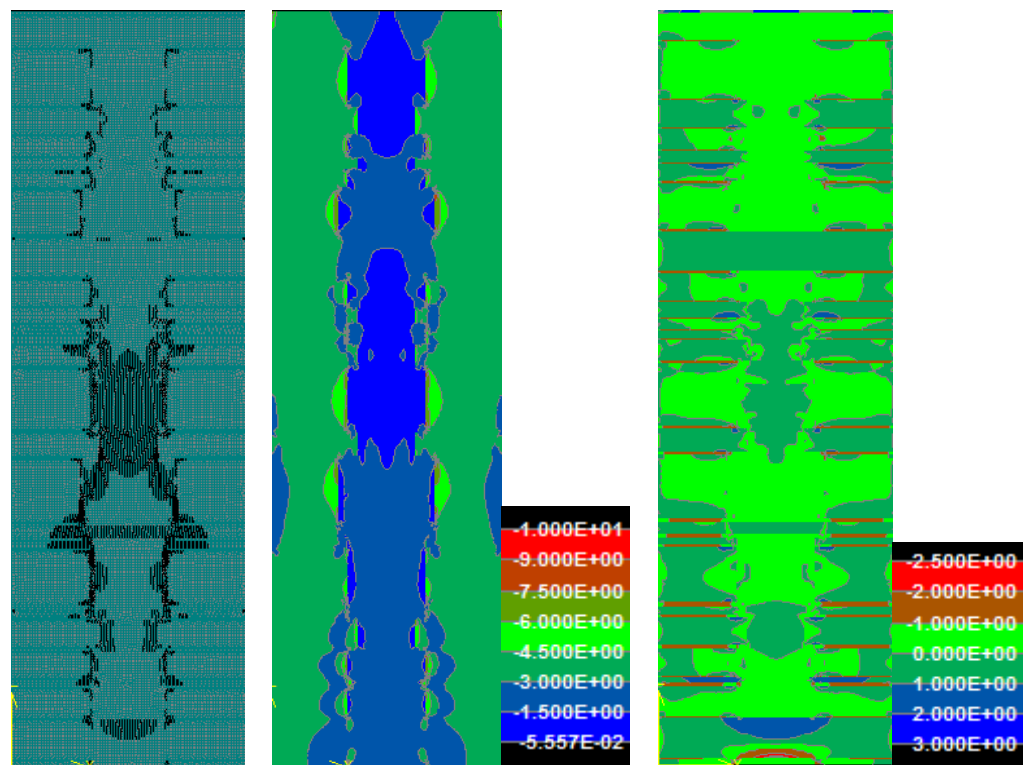


Fig. B.4 - Cracks pattern, vertical and horizontal stress distribution in Sectional Straight Wall 1 (Last Step)

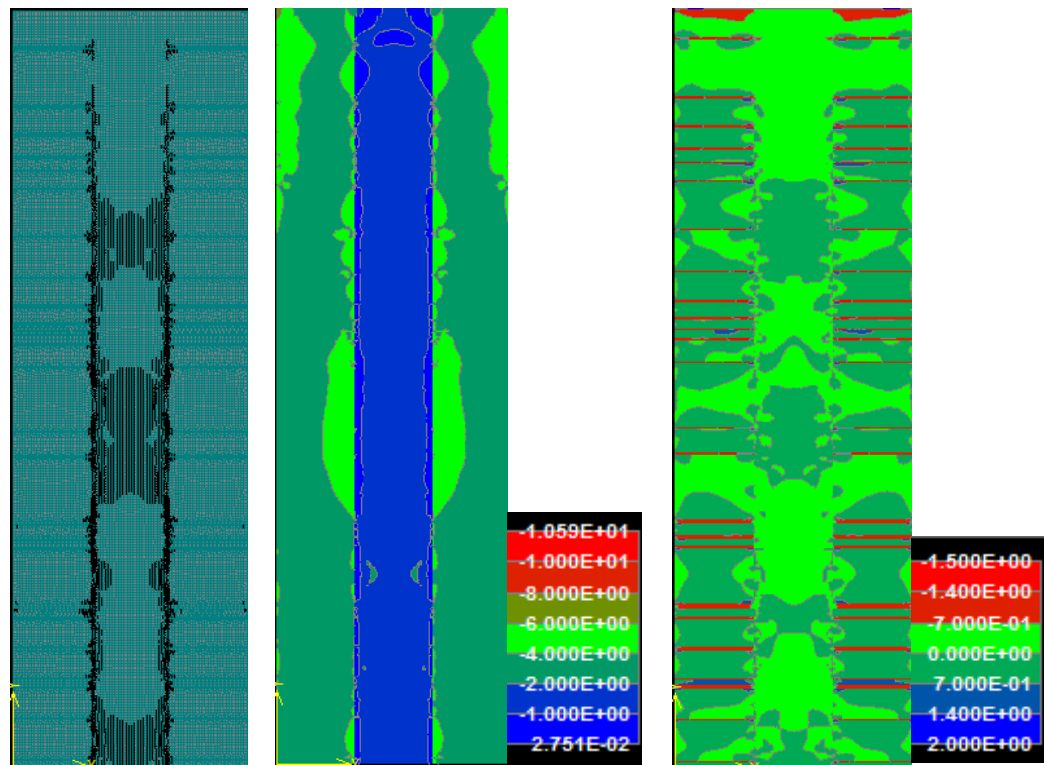


Fig. B.5 - Cracks pattern, vertical and horizontal stress distribution in Sectional Connection Wall 2 (Last Step)

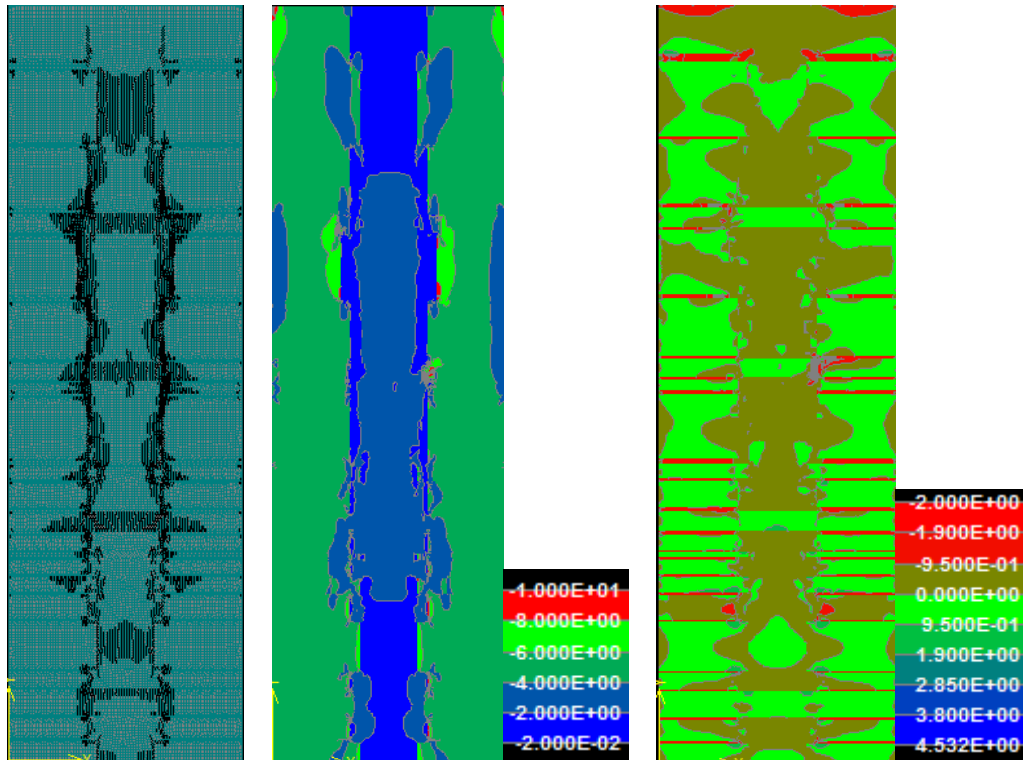


Fig. B.6 - Cracks pattern, vertical and horizontal stress distribution in Sectional Straight Wall 2 (Last Step)

