Czech Technical University in Prague

Faculty of Civil Engineering

THE CURRENT STATE OF SELECTED CHURCHES OF THE BROUMOV GROUP
NUMERICAL ANALYSIS OF WALL FAILURES

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Master's Thesis

Prague, 2020

Declaration

I declare that I have developed and written the Master Thesis completely by myself and have not used sources or means without declaration in the text. Any thoughts from others or quotations are clearly marked. All the materials I have used are listed in the references.

....
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III. ASSIGNMENT RECEIPT

I declare that I am obliged to write the Diploma Thesis on my own, without anyone's assistance, except for provided consultations. The list of references, other sources and consultants' names must be stated in the Diploma Thesis and in referencing I must abide by the CTU methodological manual "How to Write University Final Theses" and the CTU methodological instruction "On the Observation of Ethical Principles in the Preparation of University Final Theses".

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I. Abstract

The Broumov Group of Churches have distinguished place in Baroque architecture. All the churches were built in the same period (in the 18^{th} century) in the villages of Broumov region, by the same family of architects (Dientzenhofers) under the influence of abbot, Otmar Zinke.

In my diploma thesis I concentrated on the analysis of bearing capacity of the walls of two churches (namely the church of St. Anna in Vižňov and St. Barbara in Otovice). Both churches are in poor technical condition due to poor drainage of rainwater, which is mainly caused by insufficient maintenance. The walls were significantly damaged by rain and frost. The plaster fell, the exposed stones were degraded and the mortar quality deteriorated. Since the wall is no longer protected by mortar against influences, its load-bearing capacity is consequently reduced.

The focus of my work was analysis of bearing capacity of masonry. Using multilevel modeling of masonry composite I analyzed its load-bearing capacity. A micromodel of masonry was created in which all the elements of its composition, various types of stones, sometimes even bricks connected by lime mortar, were contained. Using the ATENA software of Červenka Consulting it was calculated the homogenized load-bearing capacity of the extracted masonry block, both in compression, in tension and shear. The results were plotted in graphs and arranged in tables. The composition of the micromodel together with the FEM network is also presented in the pictures.

The Broumov Group of Churches plays an extraordinary role in the cultural heritage of the region and the analysis should contribute to its sustainability. Successful reconstruction and rehabilitation will require further analysis and monitoring of the current situation.

Keywords: Baroque, Broumov group of churches, masonry, bearing capacity of walls, micro-modeling, non-linear and quasi-fragile numerical models

II. Souhrn

Broumovská skupina kostelů má v barokní architektuře výjimečné postavení. Všechny kostely byly postaveny pod vlivem opata, Otmara Zinkeho, v krátkém časovém rozmezí (počátkem 18. století), ve vesnicích Broumovského regionu převážně jedinou rodinou architektů (Kryštofem a Kiliánem Ignácem Dientzenhoferovými).

V mé diplomové práci jsem se soustředil na analýzu únosnosti stěn dvou kostelů (jmenovitě se jedná o kostel sv. Anny ve Vižňově a sv. Barbory v Otovicích). Oba kostely jsou ve špatném technickém stavu, díky špatnému odvodu srážkových vod, které je způsobeno především nedostatečnou údržbou. Vlivem dešťů a působením mrazu došlo k výraznému poškození stěn. Docházelo k opadávání omítky, degradaci obnažených kamenů a zhoršování kvality malty. Jelikož nosná stěna není nadále chráněna maltou před vlivy dochází následně ke snížení její únosnosti.

Těžištěm mé práce byla analýza únosnosti zdiva. Pomocí víceúrovňového modelování kompozitu zdiva jsem analyzoval jeho únosnost. Byl vytvořen jakýsi mikromodel zdiva, ve kterém byly obsaženy všechny prvky jeho skladby, různé druhy kamenů, místy i cihel vzájemně propojené vápennou maltou. Pomocí software ATENA Červenka Consulting jsem vypočetl homogenizovanou únosnost vyjmutého bloku zdiva, jak v tlaku, tak i v tahu a smyku. Výsledky jsem vynesl do grafů a uspořádal přehledně do tabulek. Skladba mikromodelu spolu se sítí MKP je též představena v obrázcích. do tabulek.

Broumovská skupina kostelů sehrává mimořádnou úlohu v kulturním dědictví regionu a provedená analýza by měla přispět k její udržitelnosti. Úspěšná rekonstrukce a rehabilitace bude vyžadovat další analýzu a monitorování současného stavu.

Klíčová slova: barok, Broumovská skupina kostelů, zdivo, únosnost stěn, mikromodelování, nelineární a kvazikřehké numerické modely

III. Introduction

1. Broumov region-geography, history, climatic conditions

The Broumov region (also called Broumovsko) lays in the northeast of Bohemia, in the region of Hradec Králove, which belongs to Náchod district. The history of Broumovsko is dated to 1256 A.C., when it was expelled from Klodzko (Kladsko) and it was taken in the administration of the Benedictine order from Břevnov. The Benedictine order has played a very important role in the development of the whole region. Since 1991, Broumovsko has been a protected landscape area. The Broumovsko protected landscape area is made by two different geological formations, by the Broumov basin and Police high lands. The area of the Broumovsko protected landscape area is 430 km² (1). The Broumov basin sprawls between the Javoří mountains and Broumov wall. The Broumov region belongs to the geological complex of Intra-Sudeten basin (2).

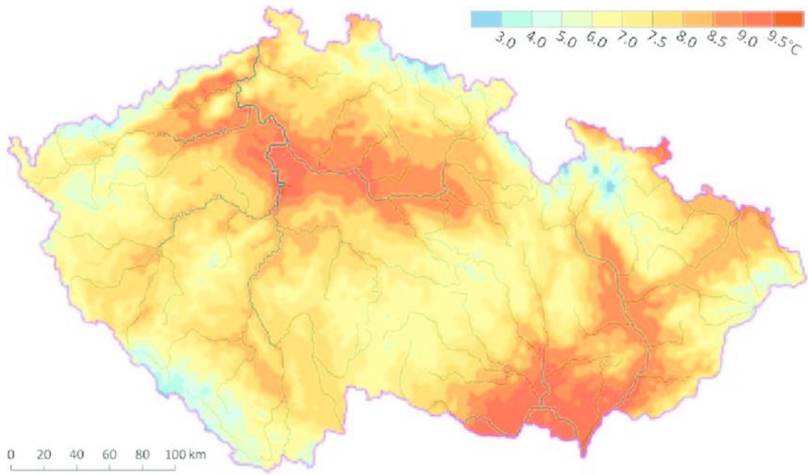
Picture 1: Broumovsko Protected Landscape Area

Source: CC BY-SA 2.5, https://commons.wikimedia.org/w/index.php?curid=283253

The climate of Broumov region is quiet moderate. The annual average temperature of the region is 6,9 °C. The coldest month is January (average temperature -3.9 °C) and the hottest July (average temperature 16.2°C). The average annual precipitation is cca. 640 mm.

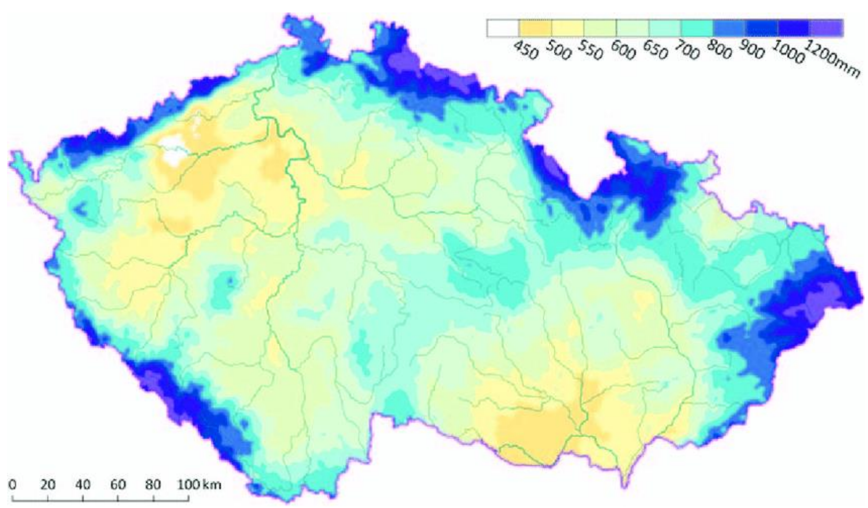
Picture 2: Broumovsko – red area

Source: https://en.mapy.cz/zakladni?x=16.2381262&y=50.5699774&z=8&source=area&id=37



Picture 3: Annual average temperature in the Czech republic

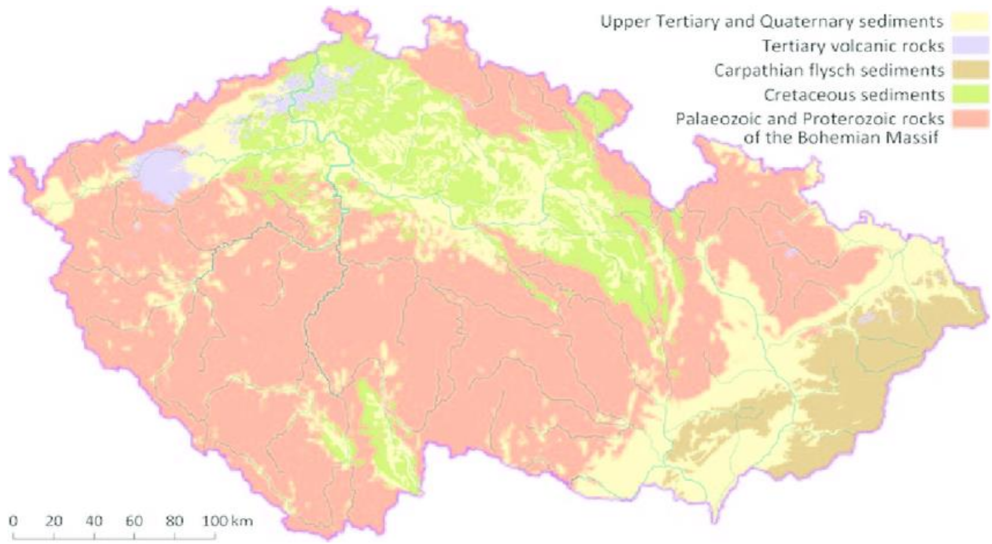
Source: CHYTRÝ, Milan. Vegetation of the Czech Republic: diversity, ecology, history and dynamics. *Preslia*. 84(3), 2012, 427–504.



Picture 4: Annual precipitation in the Czech republic

Source: CHYTRÝ, Milan. Vegetation of the Czech Republic: diversity, ecology, history and dynamics. *Preslia*. 84(3), 2012, 427–504.

Sediments and volcanic rocks create the bedrock of Broumov formation. The surface is created by deluvial sediments (colluvial deposits), fluvial sediments and by their combinations (3).



Picture 5: Geological map of the Czech republic

Source: CHYTRÝ, Milan. Vegetation of the Czech Republic: diversity, ecology, history and dynamics. *Preslia*. 84(3), 2012, 427–504.

2. The city of Broumov-history

The colonization of the Broumov region can be dated back to the Old Stone Age. Evidence, that hunters were living in this area 40 to 80 thousand years B.C. (Palaeolithic period) was founded by archaeologists. Because of the cold climate of this area, practically it was uninhabited until Early Bronz age (1000 B.C.). One of the most important periods of colonization was in the 13th century, when was inhabited by German inhabitants. The first villages of the region were created in this period.

In 1213 King Přemysl Otakar I assigned Broumov, a rocky headland in northeast Bohemia, to the Benedictine order of Břevnov (4).

The city of Broumov (Braunau) is for more than 700 years agricultural, cultural and administrative center of the region (5) and it is the starting point of the most of the touristic paths too. As it was already mentioned, the Benedictine order had a very important role in the development of the region and of the city of Broumov too.

Broumov was first mentioned as a village with a market in 1256. King Přemysl Otakar II granted to the village the privilege to produce cloths. King Charles IV, in 1348 has granted the same privileges to Broumov, as they had the citizens of the royal towns (for example Kladsko a Hradec Králove). This act integrated the Broumov region into Bohemia (2).

The city suffered several fires which resulted in the disappearance of the original medieval buildings, except the cemetery church of Virgin Mary. The castle was burned down too and it was restored in 1305. In the middle of the 14th century the castle was rebuilt to a fortified monastery. The reconstruction of the town wall has finished before 1380.

As the city was becoming wealthier, the people were starting to use for the construction of their homes better materials, as stone. In the great fire the most of the city, including the monastery was destroyed. The monastery was reconstructed in Renessaince style in 1549.

Picture 6: Broumov Monastery

Source: Author: Vlach Pavel, available on: https://commons.wikimedia.org/w/index.php?curid=68443088

Under the leading of abbots Tomáš Sartorius (1663-1700) and Otmar Zinke (1700-1738), the city has reached a very good economic status. This led to further construction and reconstruction activities (the damages were caused by the Thirty Years War), new churches were built, residences, but also the monastery underwent reconstruction.

Christoph Dientzenhofer has designed the monasterial grammar school and the pharmacy. The best Baroque architect in Bohemia, Kilian Ignaz Dientzenhofer, has made the plans of radical reconstruction of the monastery, which was realized between 1728-1738.

After the Silesian war, Kladsko and Silesia were annexed by Prussia. This led to economic disaster of whole the region, which resulted in declination of the building activities. Moreover, in 1757 and 1779 the monastery was damaged by fire.

A rail road was opened between Choceň to Broumov in 1875 (2). The town was further suffered by World Wars, the historic value of most of the buildings were disturbed. The last years, thanks to privatization, the appearance of the city is slowly rehabilitated (6).

Inhabitants In Broumov (2):
1869: 5701 inhabitants
1910: 11846 inhabitants
1950: 6713 inhabitants
1991: 8076 inhabitants

2. 1. Baroque architecture

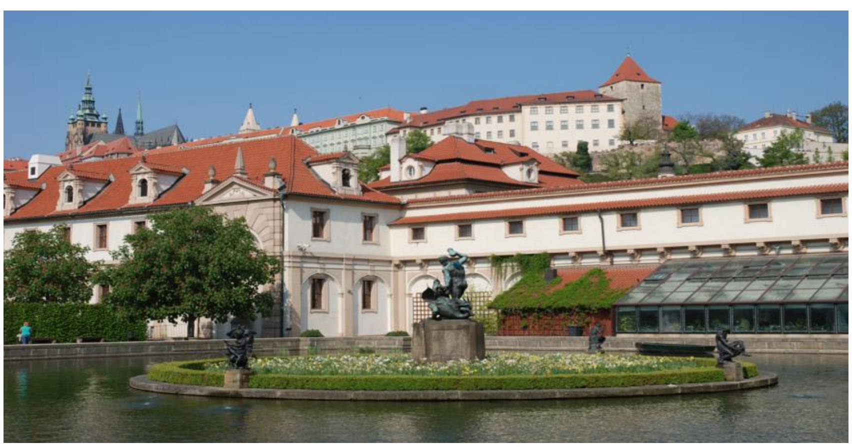
Baroque art is highly ornate style of not only architecture, but music, painting, theater and sculpture too. The word baroque originates from Portugese word „pérola barocca“, which means pearl of unordinary shape, but in the Medieval logic was meaning something pompous, comic too (7). The aim of the baroque art was to cause emotions, to impress.

Baroque architecture started in the 17th century in Italy. It is characterized by very decorative style (8). It was mainly used in churches and palaces. Light had very important role too. Oval and elliptic shapes becoming the ideal shapes of baroque architecture (but quadrangular, octagonal shapes and greek cross were very popular too). The most famous buildings of Baroque style are for example Versailles palace in France, or the St. Peter`s Basilica in Rome.

In the Czeh republic Baroque is considered to be unique due to its extensiveness. Leading architects of Czech Baroque were Christoph and Kilian Ignaz Dientzenhofers (father and son). Other important Czech architects of this period are Jan Blažej Santini Aichel, Giovanni Battista Alliprandi a František Maximilián Kaňka. Baroque prefers asymmetric forms, arches, curvatures, spacially developed gestures, perspectives. An important element of baroque art is movement.

Between the most important Baroque buildings in the Czech republic belonging for example Valdštejnský palác in Prague, which was the first Baroque palace in the central Europe, Matthias`s gate of the Prague castle or church of Our Lady Victorious.

Unique place in Baroque architecture have the Broumov Group of Churches.



Picture 7: Valdštejnský palác in Prague

Source: Author: Agnete, available on: https://creativecommons.org/licenses/by-sa/4.0

3. The Broumov Group of Churches

Broumov Group of Churches are a group of churches, which were built in villages of Broumov region in relatively short period of time, in the 18th century. All of the churches were built by the same family, by the Dientzenhofers, under the leading of the abbot of the region, Otmar Zinke (only the bell tower of the St. Martin`s and St. George`s Churches in Martínkovice were built by Martin Allio).

Because of the bad condition of the old wooden churches, which were at the end of their service life and the need of very demanding reconstruction of them, the abbott has decided, that all the churches in the villages of the region, which were belonging to the Benedictine order, will be replaced by new ones. In Bezděkov, where have not been before a church, was even built a new one.

Christoph Dientzenhofer was hired by Otmar Zinke, after the abbot has terminated the contract with another builder, P.I Bayer, in 1709. One of his most famous works are the Church of St. Nicholas in Prague (finished by his son, Kilian Ignaz) and the Monastery of Břevnov (Prague).

There is evidence for quarrying stone for construction of churches from Vernéřovice and Božanov from 1780.

Based on planes of Christoph Dientzenhofer were built churches in the following villages by his son Kilian Ignaz Dientzenhofer:

Name of the Church	Years of construction	Localization
St. Michael's Church	1719-1720	Vernéřovice
St. Jacob's Church	1720-1730	Ruprechtice
St. Barbara's Church	1725-1726	Otovice

Table 1. Churches bulit by Kilian Ignaz Dientzenhofer acording to the plans of his father, localization, years of construction

According to the plans of Kilian Ignaz Dientzenhofer were built the churches in the following villages:

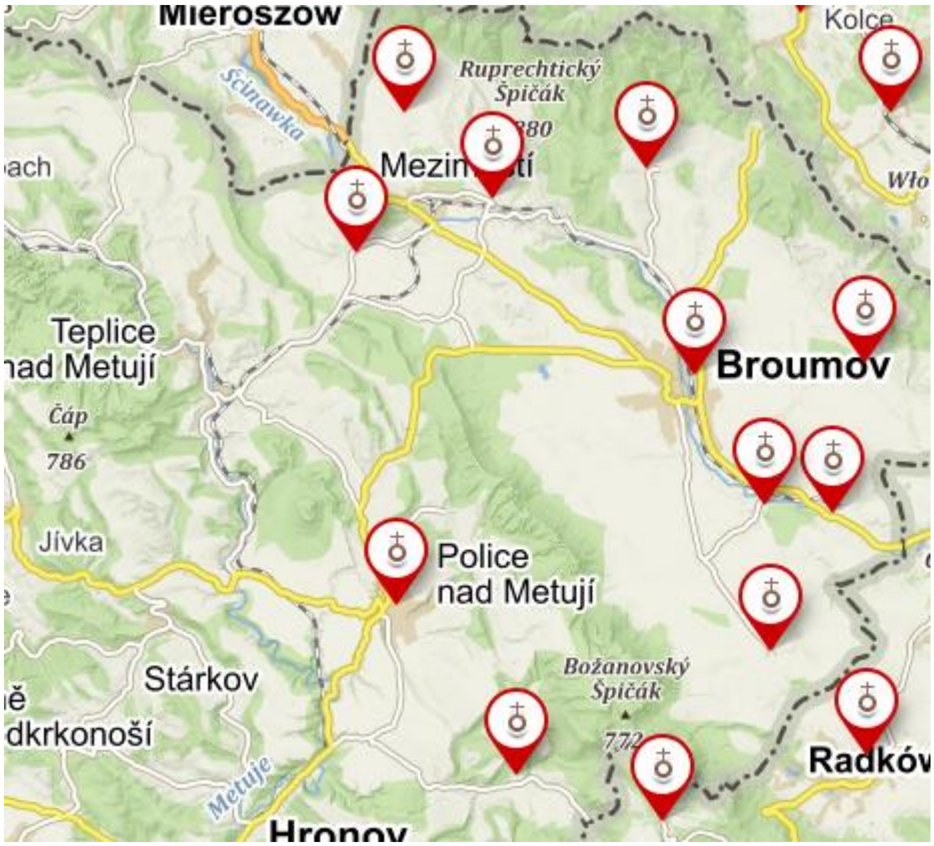
Name of the Church	Years of construction	Localization
St. Anna’s Church	1724-1728	Vižňov
All Saints Church	1722-1723	Heřmánkovice
St. Prokop’s Church	1724-1727	Bezděkov nad Metují
St. Marketa’s Church	1726-1730	Šonov
St. Mary Magdalene’s Church	1737-1740	Božanov

Table 2. Churches bulit by Kilian Ignaz Dientzenhofer, localization, years of construction

The bell towers of St. Martin’s and St. George’s Churches in Martínkovice were built by Martin Allio (9).

All the churches are single-nave with bell towers (except of St. Barbara’s Church). They had to be simple, cheap and feasible. The abbot financed only the 50% of the cost of the construction, the rest of the cost was founded from gatherings. Except from this, the lower level of masons had to be taken into consideration too.

They were building a continuous ring beam without big differences in elevations in connected buildings. The roof of the churches is steep hip, competing with the height of towers for static (uniform loading and binding of construction), economic (cheaper) and climatic reasons (rain, snow load). The original roofs were red and shingled. In the curse of history, the roofs were replaced by more modern materials.



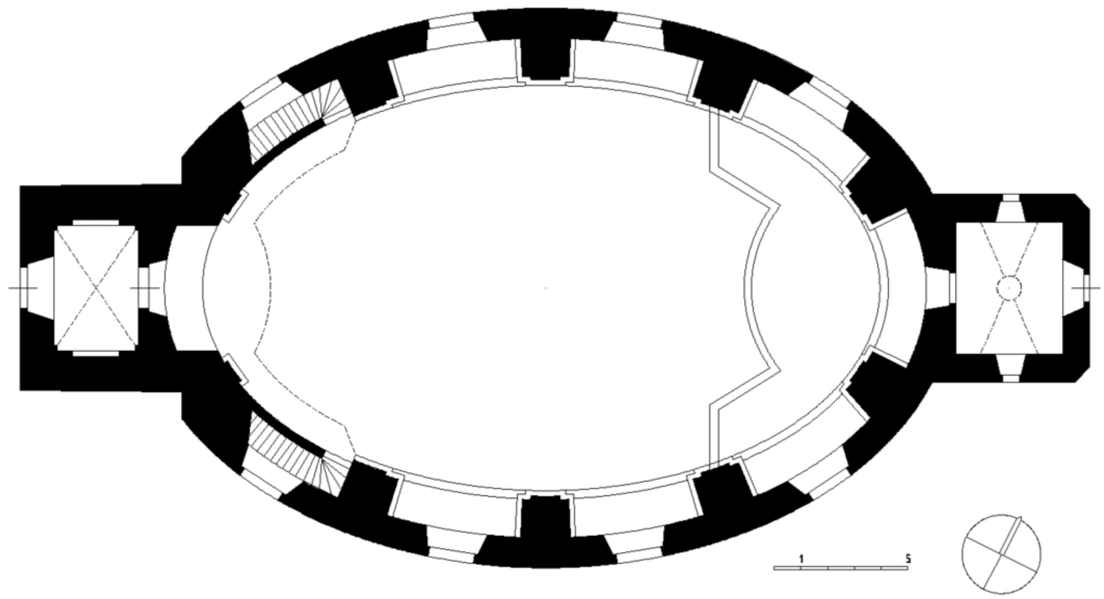
Picture 8: Localization of the Broumov Group of Churches on the map

Source: https://mapy.cz/zakladni?x=16.3473358&y=50.6018133&z=11&q=kostely%20v%20regionu%20broumov

3.1. St. Michael's Church in Vernéřovice (1719-1720)

Kilian Ignaz Dientzenhofer has built this church according to the plans of his father in the village of Vernéřovice. The church was finished in 1720, but the indoor equipment was completed 2 years later, in 1722.

The nave of the church is 26 meters long and 16 meters wide, with 10 pillars (2). The ceiling is straight. The nave is ellipse-shaped. The sacristy is 4 meters long and 5 meters wide. The choir protrudes on 3 arches into the nave and is accessible from the nave by two wooden staircases. The main entrance is under the tower (12, 2). Worship is held every Sunday.



Picture 9: Floor plan of St. Michael's Church in Vernéřovice

Source: Free work, avaliable on: https://commons.wikimedia.org/w/index.php?curid=10765252

Picture 10: St. Michael's Church in Vernéřovice

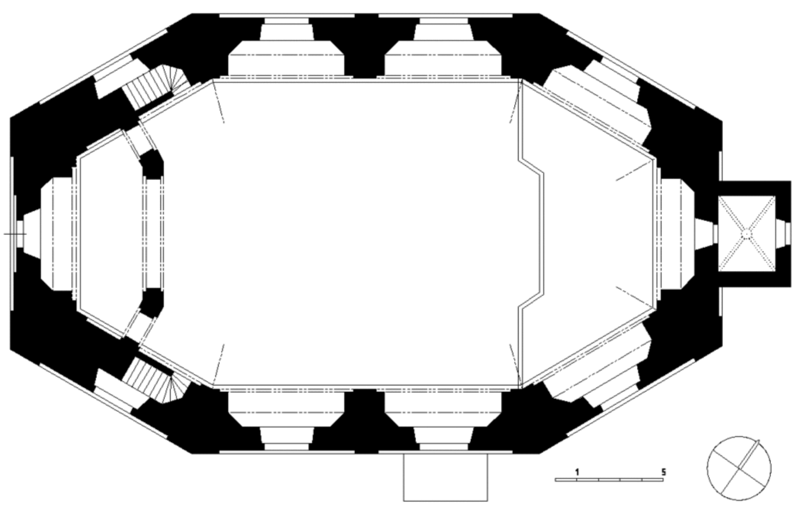
Source: Jiří Novák, available on: https://commons.wikimedia.org/w/index.php?curid=21724933

3.2. St. Jacob's Church in Ruprechtice (1720-1730)

The church was built by Kilian Ignaz Dientzenhofer according to the plans of his father in the village of Ruprechtice, between 1720 and 1730 (10), instead of a wooden church, which was mentioned already in 1386. The church stands in the middle of the cemetery.

The church is single-nave with octagonal floor plan divided by wide window niches. The church is 30,6 meters long and 16 meters wide (2).

Near to the church is a Renaissance bell tower from the 16th century (13). Worship is held twice a week.



Picture 11: Floor plan of St. Jacob's Church

Source: Free work, available on:

https://cs.wikipedia.org/wiki/Kostel_svat%C3%A9ho_Jakuba_V%C4%9Bt%C5%A1%C3%ADho_(Ruprechtice)#/media/Soubor:Kostel_svat%C3%A9ho_Jakuba_V%C4%9Bt%C5%A1%C3%ADho_(Rupr%C5%A1%C3%ADho_(Ruprechtice)#/media/Soubor:Kostel_svat%C3%A9ho_Jakuba_V%C4%9Bt%C5%A1%C3%ADho_(Ruprechtice),_p%C5%AFdorys.GIF

Picture 12: St. Jacob's Church in Ruprechtice

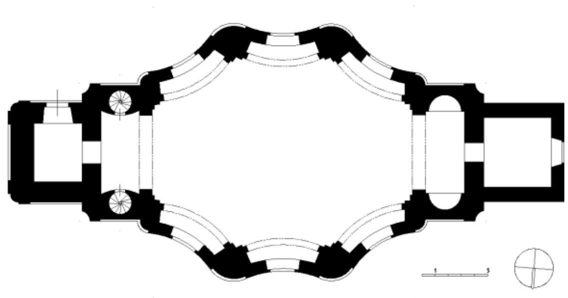
Source: Mercy from Wikimedia Commons, avaliable on:https://cs.wikipedia.org/wiki/Kostel_svat%C3%A9ho_Jakuba_V%C4%9Bt%C5%A1%C3%ADho_(Ruprechtice)#/media/Soubor:Ruprechtice_-_kostel_svat%C3%A9ho_Jakuba_V%C4%9Bt%C5%A1%C3%ADho.JPG

3.3. All Saints Church in Heřmánkovice (1722-1723)

The church was built by Kilian Ignaz Dientzenhofer. The foundation stone in 1722. Raw building was completed and consecrated in 1723 and the next three years there were still works on the completion and finishing of the interior.

The church is single-nave. The nave is 25.7 meters long and 10 meters wide (2). The chancel is 8 meters long and 2.8 meters wide. The floor plan is elongated octagon, which is extended on the one side by sacristy and on the other side by the vestibul and tower. The ceiling of the church is decorated with a view to the heaven, in the semi-circular fields there are painted images of the four evangelists (14). The tower is prismatic with rounded corners (15).

The church is in the middle of the cemetery, on a small hill.



Picture 13: Floor plan of All Saints Church in Heřmánkovice

Source: Free work, available on:

https://upload.wikimedia.org/wikipedia/commons/3/3d/Kostel_V%C5%A1ech_svat%C3%BDch_%28He%C5%99m%C3%A1nkovice%29%2C_p%C5%AFdorys.gif

Picture 14: All Saints Church in Heřmánkovice

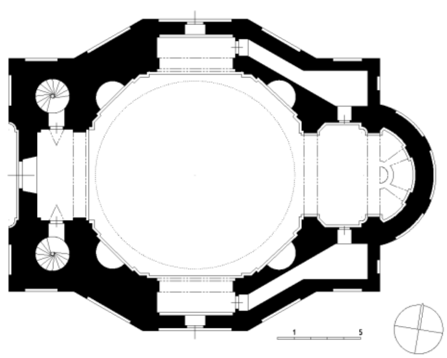
Source: Author: Anaj7, available on:

https://upload.wikimedia.org/wikipedia/commons/6/6f/He%C5%99m%C3%A1nkovice%2C_kostel_V%C5%A1ech_svat%C3%BDch_%2803%29.jpg

3.4. St. Prokop’s Church in Bezděkov nad Metují (1724-1727)

The church was built by Kilian Ignaz Dientzenhofer. The church of St. Prokop is the dominant of the village in Bezděkov nad Metují and it is also a part of the village embleme from 1999.

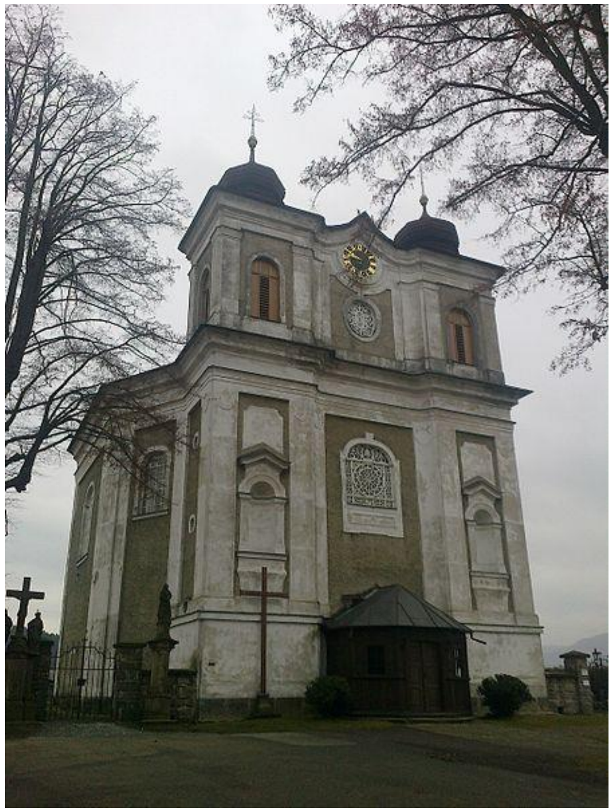
The church is octagonal, with small chapels on the side (dept of th chapels is diagonally 2.5 meters and transversally 5 meters). The nave is 16.5 meters long and 12.2 meters wide (2). The presbytery is semicircular. The church has two robust towers. A mirror vault spans the central space. On the first floor is an open semicircular gallery (16).



Picture 15: Floor plan of St. Prokop’s Church in Bezděkov nad Metují

Source: Free work, available on:

https://upload.wikimedia.org/wikipedia/commons/c/c5/Kostel_svat%C3%A9ho_Prokopa_%28Bezd%C4%9Bkov_nad_Metuj%C3%AD%29%2C_p%C5%AFdorys.GIF



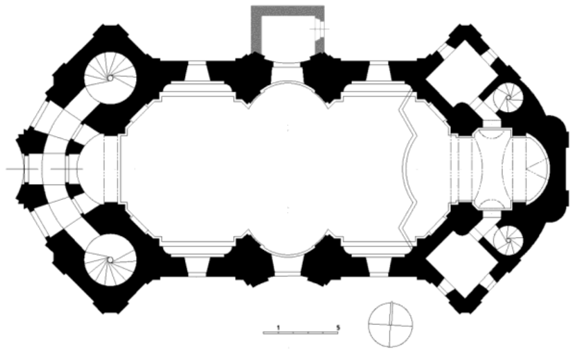
Picture 16: St. Prokop's Church in Bezděkov nad Metují

Source: Author: Autor: Addvisor, available on: https://commons.wikimedia.org/w/index.php?curid=22914912

3.5. St. Marketa's Church in Šonov (1726-1730)

The church is on the hill in the cemetery of Šonov. From the tower of the church is seen the Broumov monastery.

The nave is rectangular with symmetric spaces, on the east is a sacristy and a depository. On the west side are the towers, which are placed diagonally. The nave is 25 meters long and 12 meters wide. The chancel is 5 meters long and 4 meters wide. The roof is gabled (2). The church has 6 entrances. Worships in the church are not held.



Picture 17: Floor plan of St. Marketa's Church in Šonov

Source: Free work, available on:
https://cs.wikipedia.org/wiki/Kostel_svat%C3%A9_Mark%C3%A9ty_(%C5%A0onov)#/media/Soubor:Kostel_svat%C3%A9_Mark%C3%A9ty_(%C5%A0onov),_p%C5%AFdorys.GIF

Picture 18: St. Marketa's Church in Šonov

Source: Author: Daniel Baránek, available on: https://commons.wikimedia.org/w/index.php?curid=4745208

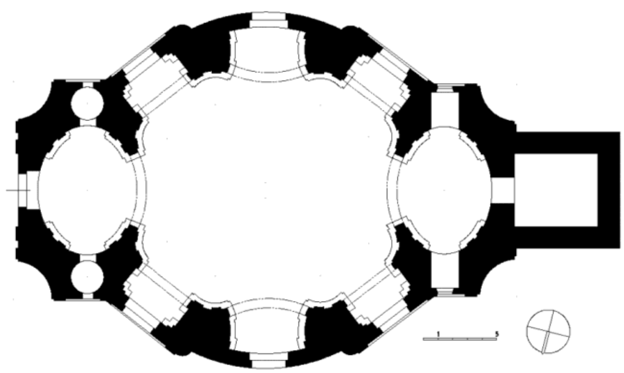
3.6. St. Mary Magdalene’s Church in Božanov (1737-1740)

Božanov is one of the oldest villages of Broumov area. The original wooden church was built in 1253 and it was just 40 meters from the present one, which is the church of St. Mary Magdalene (17).

This is the only one from the Broumov group of churches, which was built by Zinke`s successor, Benn Löbel (2).

The construction material is quarry stone. The nave of the church is circular, on the south end is connected with rectangular chancel. The lenght of the nave is 16.8 meters and the width is 10 meters. The size of the chancel is 6.5 meters length and 10 meters width.

It is the only one from the Broumov Group of Churches, which is built by complicated walled baldachin vaults. The church has been completely reconstructed, it has new roof (slate) (2).



Picture 19: Floor plan of St. Mary Magdalene’s Church in Božanov

Source: Free work, available on:
https://cs.wikipedia.org/wiki/Kostel_svat%C3%A9_M%C3%A1%C5%99%C3%AD_Magdaleny_(Bo%C5%BEanov)#/media/Soubor:Kostel_svat%C3%A9_Marie_Magdal%C3%A9ny_(Bo%C5%BEanov),_p%C5%AFdorys.gif

Picture 20: St. Mary Magdalene's Church in Božanov

Source: Author: Petr1868, available on: https://commons.wikimedia.org/w/index.php?curid=2801896

In my work I will focuse on two churces:

3.7. St. Anna's Church in Vižňov (1724-1728)

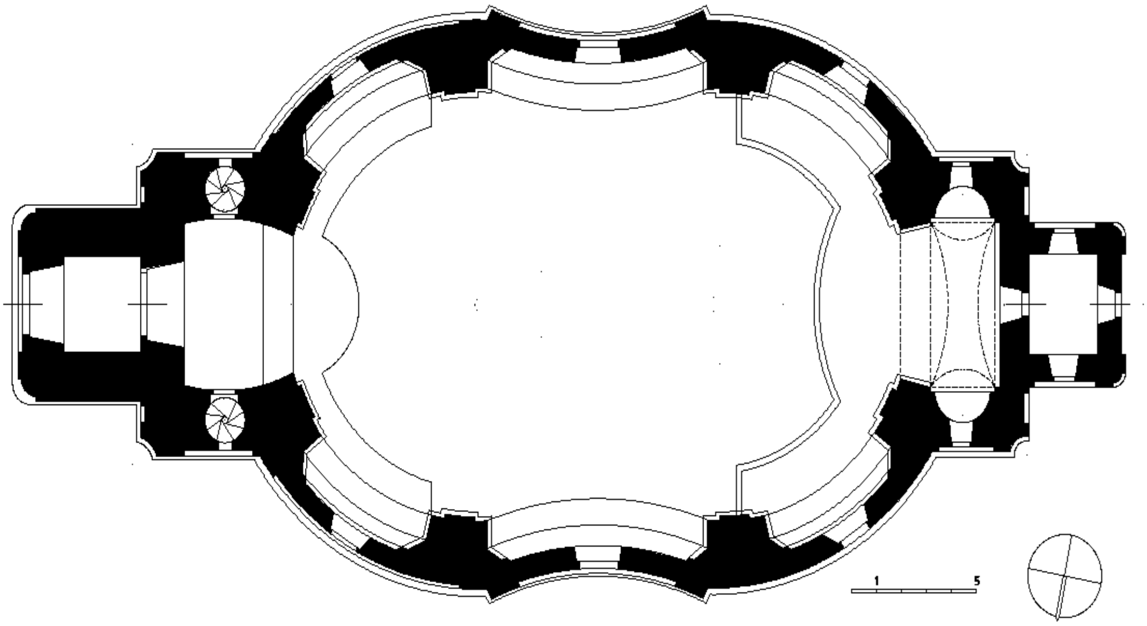
Vižňov is a village, which was established in the 13th century. Since 1434 it is under the Broumov monastery administration. In 1991 it had 3009 inhabitants (2)

The St. Anna`s Church in Vižňov is a Roman Catholic church and belongs to the Broumov group of Baroque churches. The original wooden church in Vižňov was documented already in the 14th century (11). The construction of the new church from sandstone started in 1724 according to plans of Kilian Ignaz Dientzenhofer. The church is located in the cemetery of the village Vižňov on the hill side and it is orientated westly. The nave of the church is elliptical, on the long side is concavely broken, the short side has blunted corners. In the longitudinal axis of the church is behind the chancel a small sacristy, on the east side is a prismatic tower.

The hipped roof had originally shingle cover, which at the end of the 20th century was replaced by metal plates. The church has two entrances. The main entrance is double winged (1.8m x 3m).

The nave is 15 meters wide and 24 meters long and it has elliptical shape with 8 pillars. Between the pillars are small, five-sided shallow niches. The ceiling is beamed on high ramps.

The chancel is 3 meters x 6 meters. On the sides has semicircular niches with a glass conch.
At the time, the church is in very bad condition. The plaster is dropped from the ceiling, On the floor there is in some places moss. Worship is not held at the moment in the church.



Picture 21: Floor plan of St. Anna's Church

Source: Free work, available on: https://commons.wikimedia.org/w/index.php?curid=10237086

Picture 22: St. Anna's Church in Vižňov

Source: Author: Daniel Baránek, available on https://commons.wikimedia.org/w/index.php?curid=4783472

Picture 23: Deterioration of the external wall

Picture 24: Moss on the floor of St. Anna's Church in Vižňov

3.8. St. Barbara's Church in Otovice (1725-1726)

Otovice lays south-east from the city of Broumov on the Polish-Czech borders. It was first mentioned in the 13th century. Otovice is a small village, with 351 inhabitants (18).

The church of St. Barbara was built according to the plans of Christoph Dientzenhofer, by his son, Kilian Ignaz.

The nave of the church has oval shape, on the sides has seven semicircular chapels. The entrance is on the western facade. The church does not have tower. The interior is divided by eight pilasters. The ceilling is straight and covered by wooden shingles. The church is 22 meters long and 15 meters wide. The height of the church is 15.5m. The size of the chapels is 5.2 meters (length) X 2.5 meters (width) (2).

The whole Broumov region is rich in stone. In Otovice mining of limestone started in the 18th century and it was lasting until the II. World War, when it was definitevly stopped and it was never renewed. The church is operating regularely, worship is held each sunday.

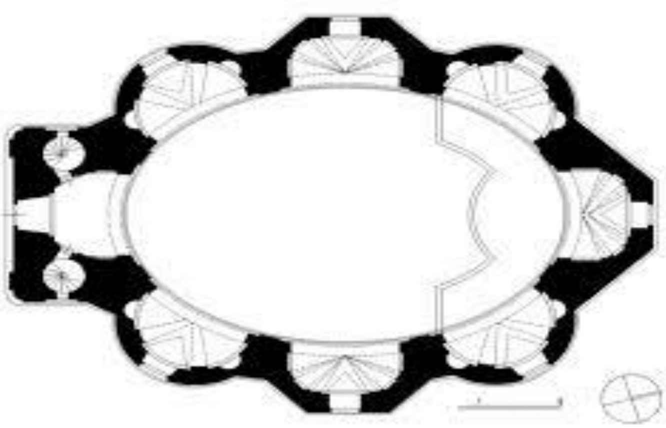


Photo 25: Floor plan of St. Barbara's Church

Source: Free work, available on:https://commons.wikimedia.org/wiki/File:Kostel_svat%C3%A9_Barbory_(Otovice),_p%C5%AFdorys.GIF#/media/File:Kostel_svat%C3%A9_Barbory_(Otovice),_p%C5%AFdorys.GIF

Picture 26: St. Barbara's Church

Source: Author: Jiří Novák, available on: https://commons.wikimedia.org/w/index.php?curid=21724643

Picture 27: St. Barbara's church i Otovice

Source: Author: Daniel Baránek, available on: https://commons.wikimedia.org/w/index.php?curid=4745054

IV. Borehole testing of the churches

Borehole testing was serving to get datas about the geological statue of the churches. The Department of Geotechnics of the Civil Engineering Faculty of ČVUT in Prague was responsible for providing and analysing the samples, which were obtained in depth of 12 meters. These results were used for soil stiffnes calculation and understanding of the soil settlement. Different types of rock (claystone, silstone, sandstones) were found under the foundation The underground water level is relatively high, water can be found in depth of 3.5 meters.

Each obtained sample was approximately 5 cm in diameter. The core of borehole was cat on sur/plus 5 cm high cylinders. The tests were arranged according to the Czech Standard respecting the required speed of load

V. Damage assesment

In order to be able to map the damage of the churches, site investigation was executed during one day visit of the site, with the representatives of department of mechanics and geotechnics together.

The site investigation was focused on:

-visual inspection of the churches

-damadge assesment (moisture, material degradation, biological degradation), including crack investigation of the walls

-measurement of the wall dimensions

-photo documentation

Sample collection was provided by the department of Geotechnics of ČVUT, as was mentioned above.

As is mentioned in the chapters 3.7 and 3.8, the size of the churches is:

a) St. Anna's Church is single-nave, which is 24 meters wide and 46.5 meters long and it has elliptical shape with 8 pillars. Between the pillars there are small, five-sided shallow niches. The ceiling is beamed on high ramps.

The chancel is 3 meters x 6 meters.

b) St. Barbara's Church is 22 meters long and 15 meters wide. The height of the church is 15.5m. The size of the chapels is 5.2 meters (length) X 2.5 meters (width).

The damage can be classified into categories: Cracks and deformation, detachment, material loss, discoloration and deposit, biological colonization.

Rising capillarity caused on both churches severe damage (more on St. Anna's Church), which led to loss of plaster and biodegradiation due to moss and fungi. The loss of the render causes, that the stones are not protected and they are exposed to weathering and other external influences directly. Because of the roof damage and insufficient drainage system, St. Anna's Church is in even worst condition due to rainwater flow, causing severe problems of soil settlement too.

The walls and the ceiling are cracked on several places. The crack of the ceiling is originated in the vault-system, used in the Broumov Group of Churches. Deformated timber elements are also a reason, why cracks are located around main trusses.

Cracks and deformation

Both churches have interor and exterior cracks. Crack is a complete or incomplete separation of the concrete into two or more parts (19) (Pictures 28-32). Burland and Day (20) provides classification of cracks based on visual damage to the walls:

Width (mm)	Category	Classification
Less than 2mm	Very slight	Aesthetic
2mm to 5mm	Slight	Aesthetic
5mm to 15mm	Moderate	Serviceability
15mm to 25mm	Severe	Serviceability
Over 25mm	Very severe	Stability

Table 3: Classification of cracks based on visual damage

Picutre 28: Cracks of the Ceiling (St. Barbara's Church)

Picture 29, 30: Cracks of the inernal wall and ceiling (St. Anna's Church)

Picture 31: Cracks, render loss and erosion of the stones (St. Anna's Church)

Picture 32: Deformation of the external wall (St. Anna's Church)

Detachment and material loss

Detachment can be blistering, bursting, delamination, disintegration, fragmentation, peeling, and scaling . Usually starts from the surface. Wind, particles, rain and their combinations plays important role on the detachment process. The exterior of the Churches are erosed mainly due to freeze and thaw cycles (Picture 31, 33).

Picture 33: Missing render of the external wall (St. Barbara's Church)

Discoloration and deposit

Discoloration means change of the color of the material surface. Depositrepresented as accumulated material, for example salt or pollution.

Picture 34: Discoloration of the external wall and missing render (St. Barbara's Church)

Moisture

Moisture can lead to damage of the plaster and renders of a building. The presence of moisture in the surfaces allows growth of moss and fungi, which causes biological degradation of the material.

Picture 35: Moisture, biological degradation - internal wall and floor (St. Anna's Church)

VI. Material properties

To obtain information about material properties we need to provide testing of the stone, which we can generally divide to non-destructive (NDT) and destructive (DT) testing.

1) NDT

Non-destructive test allows to make different inspections to evaluate the material without damaging it, so the sample can be used after the testing again. It serves to detect cracks, voids or other imperfections, but also it can help to monitor changes in the concrete in time. They are several types of NDT, each of them having their advantages and limitations.

In our cse we used from the methods of NDT

a) visual inspection
b) Schmidt rebound hammer – measures the rebound of spring-loaded mass, the rebound depends on hardness of the concrete, it has arbitrary scale from 10 to 100.

2) DT

Destructive test is carried out until the failure of the sample, so it does not allow to use sample further. The main aim is to investigate the service life and detect the weakness of design which might not show under normal working condition (Gupta, 2018). DT is cheap, fast and easier to interpret.

Sandstone was the main building material used generally for construction of the Broumov churches, as the region is rich in sandstone.

St. Anna's Church

1) Non-destructive methodes:

Schmidt hammer test was provided by Kuklík et al., in order to obtain the superficial strength of the wall structure. Two different spots were chosen to provide the test on wall size 1 meter X 1 meter. Four different types of stone were tested. Each stone sample was tested 10 times and the average value of the rebound number was determined. The average number of rebound numbers was transformed into the equivalent strength values according the formula provided by the manufacturer of the equipment.

Stone sample	Average rebound hammer value	Equivalent strength MPa
Sample 1	52.48	41.48
Sample 2	52.83	41.96
Sample 3	43.2	29.81
Sample 4	49.45	37.47

Table 4: Superficial strength of stones - top of the wall
Source: Kuklík et al., 2019 KUKLÍK, Pavel, Martin VÁLEK, Pratik GAJJAR a Jacopo SCACCO. The Basic Tasks in Evaluation of Ancient Structures Sustainability and in Estimation of Enclosure Walls Bearing Capacity. *International Journal of Structural and Civil Engineering Research*. 2019, **8**(4)

Stone sample	Average rebound hammer value	Equivalent strength MPa
Sample 1	40.7	26.99
Sample 2	50.65	39.03
Sample 3	43.9	30.63
Sample 4	53.2	42.46

Table 5: Superficial strength of stones – bottom of the wall
Source: KUKLÍK, Pavel, Martin VÁLEK, Pratik GAJJAR a Jacopo SCACCO. The Basic Tasks in Evaluation of Ancient Structures Sustainability and in Estimation of Enclosure Walls Bearing Capacity. *International Journal of Structural and Civil Engineering Research*. 2019, **8**(4)

Material type	Young's Modulus E (GPa)	Poisson's ratio ν	Tensile Strength (MPa)	Compression Strength (MPa)	Fracture Energy in Tension (N/ m)	Unit Weight (kN/m 3)
Lime Mortar	0.13	0.49	0.1	1.5	10	20
Red Sandstone	20	0.2	1.5	30	43.5	21
Grey Sandstone	13	0.2	2	20	58	21
Green Sandstone	8	0.2	1.2	12	34.8	21
Rubble Masonry	0.7	0.2	0.1	2	10	20
Steel Plate	200	0.3	-	-	-	-

Table 6: Superficial strength of stones

Source: KUKLÍK, Pavel, Martin VÁLEK, Pratik GAJJAR a Jacopo SCACCO. The Basic Tasks in Evaluation of Ancient Structures Sustainability and in Estimation of Enclosure Walls Bearing Capacity. *International Journal of Structural and Civil Engineering Research*. 2019, **8**(4)

2) DT

Sandstone destructive testing in the Broumov region it was reached the peak strength value around 40 MPa (Kuklík et al.).

St. Barbara's church

Non-destructive in situ testing (Schmidt Hammer and thermographic camera) was used to obtain the mechanical properties of stones (Bozulic, 2018). The bond and arrangement of the stones is visible in situ investigation, as the plaster is damaged on several places. Lime mortar was used as joint between the stones. The main building materials were grey sandstone, mudestone, and brick.

Material type	Young's Modulus E (GPa)	Poisson's ratio ν
Lime Mortar	2.53	0.49
Red Siltstone	11.74	0.38
Grey Sandstone	27.46	0.21
Rubble Masonry	0.6	0.2
Brick	4.7	0.25
Steel Plate	251	0.43

Table 8: Mechanical properties of the elements

KUKLÍK, Pavel, Martin VÁLEK, Pratik GAJJAR a Jacopo SCACCO. The Basic Tasks in Evaluation of Ancient Structures Sustainability and in Estimation of Enclosure Walls Bearing Capacity. *International Journal of Structural and Civil Engineering Research*. 2019, **8**(4)

VII. Micromodeling

Masonry characteristics (composite material of units and mortar) make difficult to apply numerical methods for analysis of it. To be able to make analysis by the help of micromodeling, it requires to use masonry units together with the mortar and unit-mortar interface for the analysis. Micromodeling is a very useful tool to describe masonry, as it is a very complicated material due to above mentioned characteristics, which would be very time-demanding to analyze otherwise. The joints between the masonry units represent a place, where fractures can potentially appear.

In case of micromodeling masonry units are enlarged and represented by continuous elements, masonry mortar joints are represented by discontinuous elements. The joints are represented by mortar and two interfaces (from both sides of the mortar is one interface of unit-mortar) (21).

VIII. Results

1. Determination of the compressive strength

Masonry structures are generally speaking weak in tension. The reason of their weakness is that they are composed of different materials and between them there is mortar. The mortar serves as a bond between the different materials but it is the weaker component of a masonry wall. As a result, historic masonry structures are able to effectively resist compressive forces but they are not expected to resist the tensile stresses.

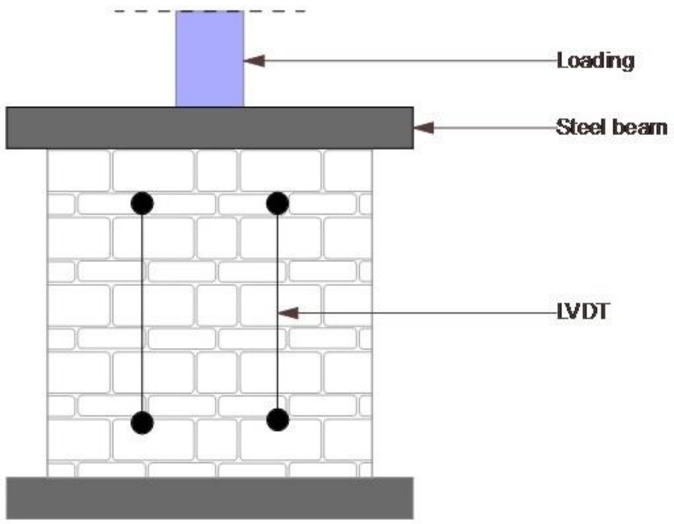


Figure 1: Typical arrangement of a uniaxial compression test of a masonry panel

In order to be able to determine the compressive strength of a building material, it is typical to perform a compression test in the laboratory. Building materials that are usually tested are natural stones, bricks, blocks of concrete, concrete cylinders and many more. In addition, a compression test can be performed on a full-size building unit as a masonry wall or it can be carried out on a smaller scale specimen in case of laboratory restrictions. Anyway, the specimen should be representative of the full-size unit.

The material properties of the components of the specimen should be as similar as possible to the material properties of the real building unit that is under investigation. In addition to that, in the case of a masonry unit, it is important that the test specimen is built according to the masonry bond pattern and the bond thickness of the original unit.

At Figure 1, it can be seen a typical arrangement of a uniaxial compression test of a masonry panel. The panel is placed between two steel beams or two steel plates, then a compressive load is applied at the top of the steel beam. The steel element is used to distribute the load to the test specimen. The test speed can be regulated by adjusting the displacement rate (mm/minute) according to the protocol of the test. As the vertical displacement increases with a constant rate and the specimen shortens, the compressive force that is applied at the top of the steel element increases too. The test continues until the failure of the specimen occurs. The vertical displacement of the panel is monitored during the test through the linear variable differential transformers. The compressive force that is applied at the top of the steel element is also monitored during the whole test by a load cell. The compressive stress is then calculated by:

$$\sigma = \frac{F}{A}$$

where F is the applied compressive force and A is the surface area of the cross section of the masonry panel before the beginning of the compression test. The engineering strain is then calculated by:

$$\varepsilon = \frac{\Delta_H}{H}$$

where Δ_H is the difference of the height of the panel and in the case of a laboratory testing, it can also be taken as the difference of the vertical length of the part of the panel that is monitored for vertical displacement (shortening). Where *H* is the initial height of the panel and in the case of a laboratory experiment, it can be taken as the initial vertical length of the part of the panel that is monitored for vertical displacement.

Compression test simulation

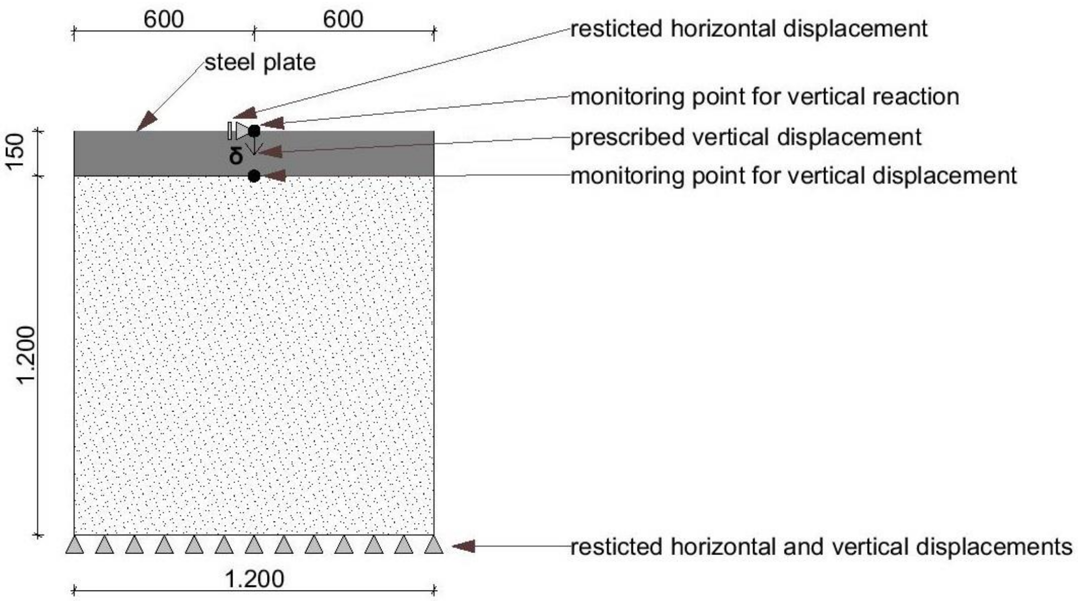


Figure 2: Numerical model arrangement for simulation of a compression test

In order to be able to determine the compressive strength of the enclosure walls of St. Barbara's church and the compressive strength of the enclosure walls of St. Anna's church, two numerical models (micro-models) were introduced, one for each church. The models were used for the simulation of a compression test performed on a masonry wall specimen.

The arrangement of the numerical models and their boundary conditions, are seen on Figure 2. The horizontal and vertical displacements at the bottom are restricted.

At the top of the steel plate, at the point where the force would be applied during a compression test (in the middle), the horizontal displacement is restricted and a prescribed vertical displacement is applied. The horizontal line between the steel plate and the masonry wall, which in this case represents the area that the surface area of the cross section of the wall and the surface area of the cross section of the steel plate meet, is considered to be fixed, so that we can take into account the effects of friction between the surfaces of the steel panel and the masonry wall.

We know that the force due to the static friction, which is located between the horizontal surface of the steel plate and the horizontal surface of the masonry panel, depends on the coefficient of static friction (μ_{static}) and the normal force (N). The coefficient of static friction relies on the materials that are in contact (for example metal on wood) and it can be obtained experimentally or it can be found in tables. As the load on the steel plate increases during a compression test, so does the normal force, which means that the force of the static friction ($F_{static} = \mu_{static} \times N$) also increases.

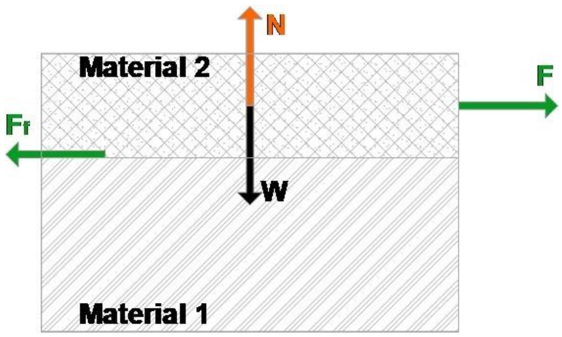


Figure 3: Static friction between 2 elements

In order to be able to monitor the vertical displacements during the nonlinear finite element analysis (simulation), a monitoring point was placed at the top of the masonry panel at the midpoint of the horizontal line between the steel plate and the masonry wall.

The monitoring point was used as linear variable differential transformers would be used in the case of a real compression test. It was applied to monitor the vertical displacement in order to be able to calculate the compressive strain and also in order to be able to control if the prescribed displacement, which was applied at the top of the steel plate, was equal to the recorded vertical displacement below the steel plate and at the top of the wall during each step of the analysis.

- **St. Anna's church - Enclosure walls**

The modelled stone masonry panel (Figure 4) that was used for the simulation of the compression test had height 1.2 meters, width 1.2 meters and thickness 1 meter. One meter is the average thickness of the enclosure walls of St. Anna's church. For the construction of the micro-model 769 joints were applied in the global X-Y coordinate system in order to be able to form the necessary macro-elements. The masonry wall and the components of the wall were represented by 210 macro-elements. The steel plate placed on the top of the wall was represented by 1 macro-element. A multi-step analysis was considered. At each step a prescribed vertical displacement increment was applied. The maximum number of iterations during the analysis have been selected as 50. The increment was considered to be 0.5 mm for all the steps of the analysis. The size of the mesh was chosen to be 1 cm.

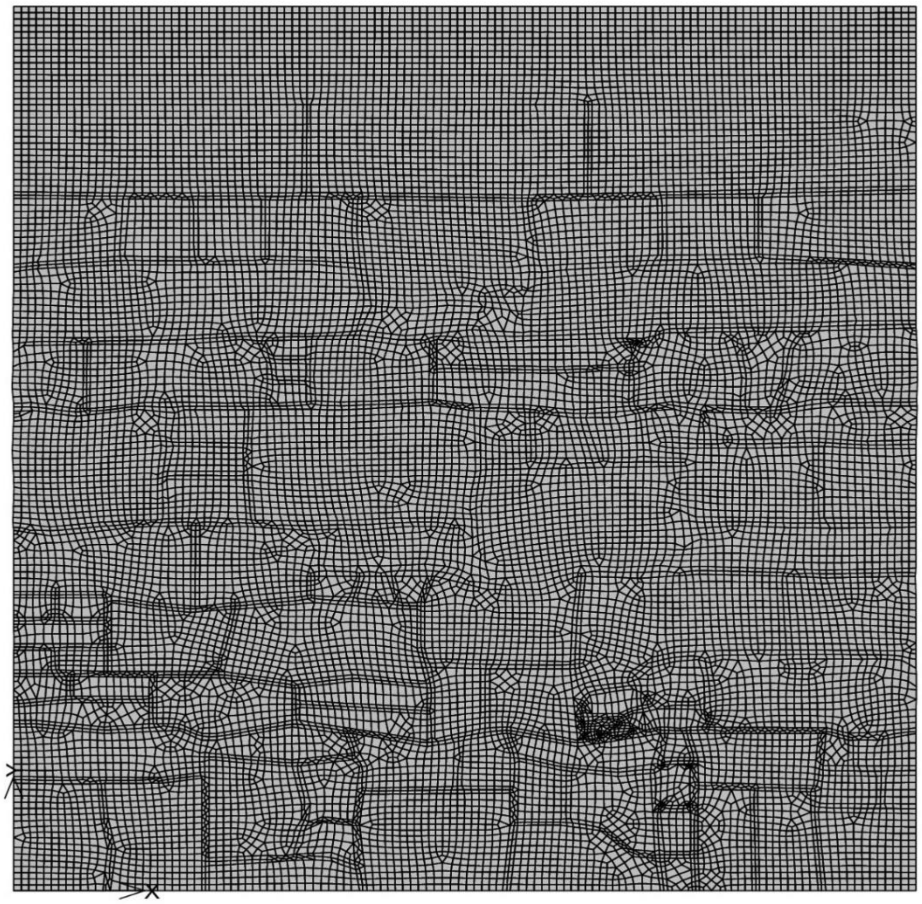
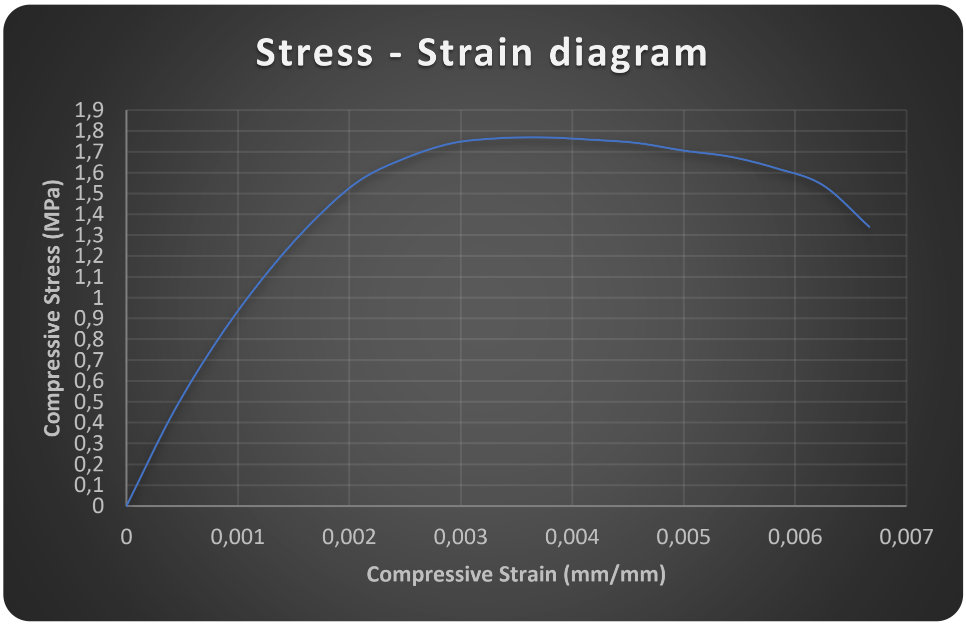


Figure 4: Micro-model of the test specimen which represents St. Anna’s church enclosure walls

Compression test simulation results:



Load step	1	2	3	4	5	6	7	8
compressive stress [Mpa]	0,446	0,809	1,111	1,363	1,559	1,669	1,741	1,765
Vertical strain [mm/mm]	0,00042	0,00083	0,00125	0,00167	0,00208	0,0025	0,00292	0,00333
Load step	9	10	11	12	13	14	15	16
compressive stress [Mpa]	1,769	1,758	1,743	1,706	1,677	1,621	1,540	1,340
Vertical strain [mm/mm]	0,00375	0,00417	0,00458	0,005	0,00542	0,00583	0,00625	0,00667

Figure 5: Stress – Strain diagram based on compression test simulation

On Figure 5 we can see the stress-strain curve of the stone masonry element that was obtained by the uniaxial compression test simulation. According to the curve, the ultimate compressive stress is 1.769 MPa which is equal to the ultimate compressive strength of the stone masonry panel. At the load step 9 of the analysis, which is the load step of the ultimate stress, the compressive strain is 0.00375, the compressive force that is applied at the top of the steel element is 2123 kN and the vertical displacement is 4.5 millimeters (Figure 31). We can estimate the modulus of elasticity of the stone masonry element by finding out the point of the curve at which the stress-strain curve is almost linear.

This occurs at the early loading steps of the analysis. If we find this point, we can assume linear elasticity and use the Hooke's Law in order to calculate the modulus of elasticity by:

$$E = \frac{\sigma}{\varepsilon}$$

Where E is the modulus of elasticity, where σ is the normal stress and where ε is the normal strain. The modulus of elasticity was estimated as 0.975 GPa (valid for the particular portion of the curve). The compressive stress at which cracks wider than 0.1mm started to develop was 1.111 MPa and the compressive stress at which cracks wider than 1mm started to develop was 1.765 MPa.

Compressive stress [MPa]	Max crack width [mm]
1.769	1.482
1.765	1.14
1.111	0.1413

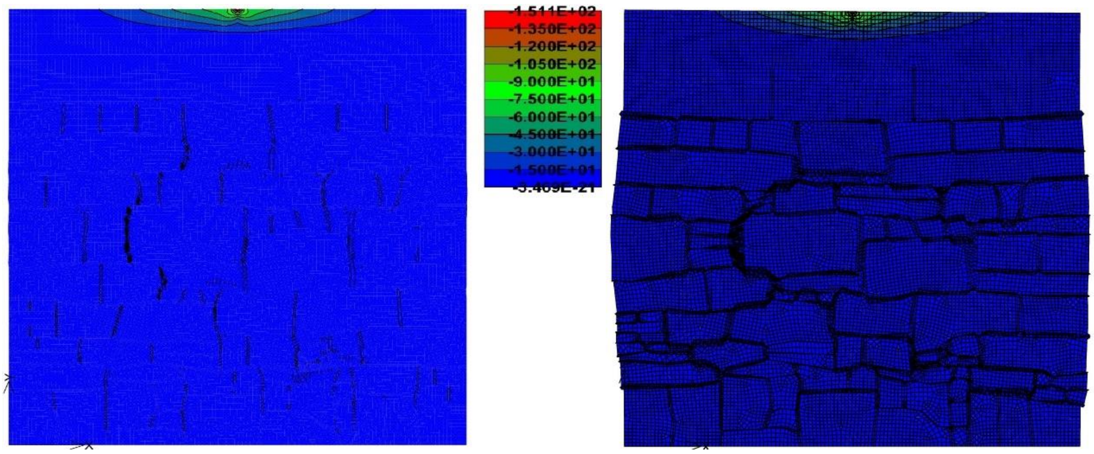


Figure 6: Minimum Principal stress in MPa and crack propagation at step of max. compression stress (Left: without mesh, undeformed shape / Right: with mesh. deformed shape)

- **St. Barbara's church - Enclosure walls**

In Figure 7 can be seen the micro-model of the stone masonry panel that was used for the simulation of the compression test. The simulation was performed, so that the ultimate compressive strength of the enclosure walls of St. Barbara's church could be determined. The masonry panel was designed with a height of 1.2 meters, with a width of 1.2 meters and finally with a thickness of 1.2 meters. The typical thickness of the enclosure walls of St. Barbara's church is 1.2 meters. The micro-model is composed of macro-elements. The macro-elements consisted of 636 joints. The stone masonry panel and the components of the panel were represented by 152 macro-elements. The macro-elements were assigned with the material properties of the element of the real wall that they represented (for example grey sandstone). The steel plate that is placed on the top of the wall was represented by 1 macro-element. The analysis consisted of multiple steps. The maximum number of iterations during the analysis have been selected as 50. At each step a prescribed vertical displacement increment was applied. The increment was considered to be 0.5 mm for all the steps of the analysis. The size of the mesh was chosen to be 1 cm.

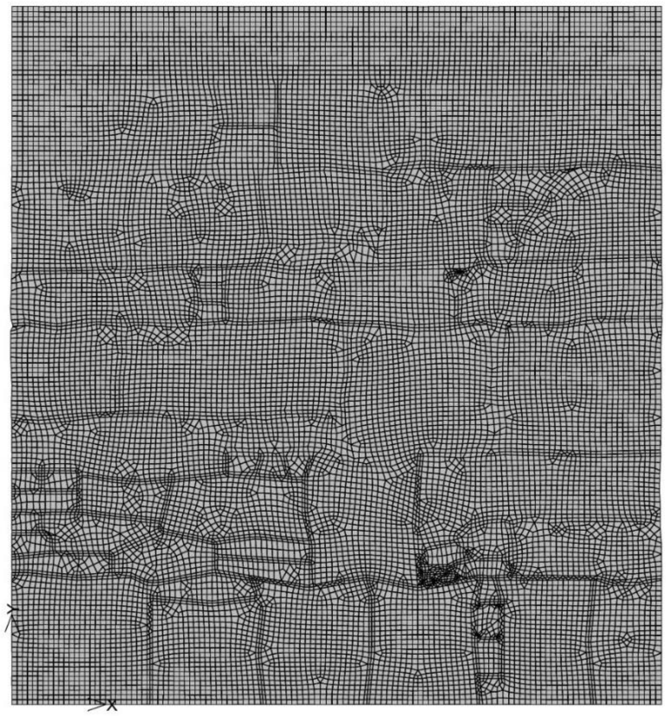
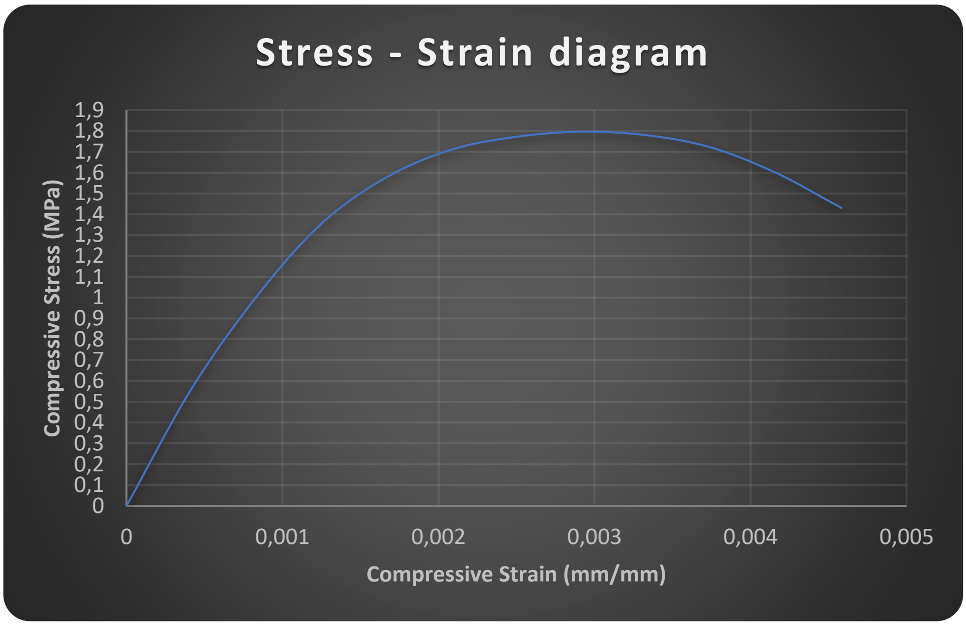


Figure 7: Micro-model of the test specimen which represents St. Barbara's church enclosure walls

Compression test simulation results



Load step	1	2	3	4	5	6
Compressive stress [Mpa]	0,5623	1,0049	1,3556	1,5778	1,7104	1,7722
Compressive strain [mm/mm]	0,00042	0,00083	0,00125	0,00167	0,00208	0,00250
Load step	7	8	9	10	11	
Compressive stress [Mpa]	1,7965	1,7799	1,7201	1,5979	1,4306	
Compressive strain [mm/mm]	0,00292	0,00333	0,00375	0,00417	0,00458	

Figure 8: Stress – Strain diagram based on compression test simulation

The stress-strain curve that was produced by the uniaxial compression test simulation, can be seen in Figure 8. By looking at the graph we can see that the maximum compressive stress is 1.7965 MPa, therefore the ultimate compressive strength of the stone masonry specimen is also 1.7965 MPa. The maximum compressive stress is achieved on the seventh step of the analysis. On the same load step, the compressive strain is 0.00292, the compressive force that is applied at the top of the steel element is 2587 kN and the vertical displacement is 3.5 millimeters (Figure 33). The modulus of elasticity of the stone masonry element is estimated as 1.206 GPa, by assuming linear elasticity at the almost linear portion of the curve.

This can be considered during the early loading steps (it is only valid for the particular portion of the curve). The compressive stress at which cracks wider than 0.1 mm started to develop was 1.3556 MPa.

Compressive stress [MPa]	Max crack width [mm]
1.7965	0.558
1.3556	0.1653

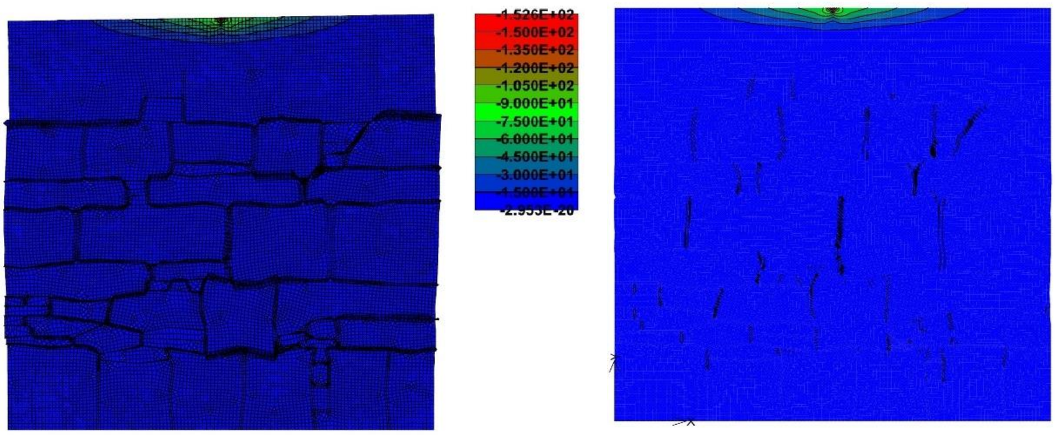


Figure 9: Minimum Principal stress in MPa and crack propagation at step of max. compression stress (Right: without mesh, undeformed shape / Left: with mesh. deformed shape)

2. Determination of the tensile strength

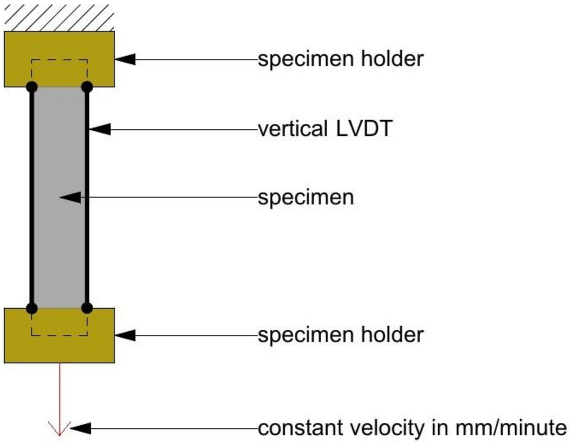


Figure 10: Basic idea of the setup of a tensile test

The basic idea of the setup of a tensile test can be seen in Figure 10. The tested specimen is positioned between the specimen holders. The ends of the specimen can have a special shape so that they can be held firmly in place during the test or they can be fixed in place by different methods. The test speed can be regulated by adjusting the velocity according to the needs of the testing personnel. As the specimen elongates at a constant rate, the tensile force that is applied on the specimen increases too. The applied load increases till the failure of the specimen. The vertical displacements during the tensile test are monitored by the linear variable differential transformers. The tensile strain is then calculated by:

$$\varepsilon = \frac{\Delta_{mean}}{H_{instument}}$$

Where Δ_{mean} is the average displacement, that it is calculated by:

$$\Delta_{mean} = \frac{LVDT_1 + LVDT_2}{2}$$

where $LVDT_1$ is the displacement recorded by the linear variable differential transformer that is installed at the left extremity of the specimen and $LVDT_2$ is the displacement recorded by the linear variable differential transformer that is located at the right extremity of the specimen. Where H is the measuring length of the linear variable differential transformers, which is the same for both transformers. The tensile force that is applied on the specimen is monitored by a load cell. The tensile stress is then calculated by:

$$\sigma_t = \frac{f_t}{A}$$

Where f_t is the tensile force that is applied on the test specimen and A is the surface area of the cross section of the specimen, which is based on the initial characteristics of the specimen. After the calculation of the tensile stress and the tensile strain, the stress strain diagram can be constructed.

Tensile test simulation

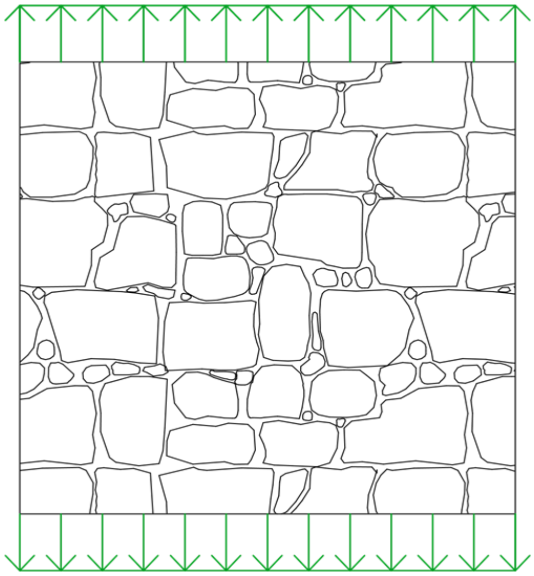


Figure 11: Theoretical model for obtaining the tensile strength of a masonry wall by the application of direct tensile stresses

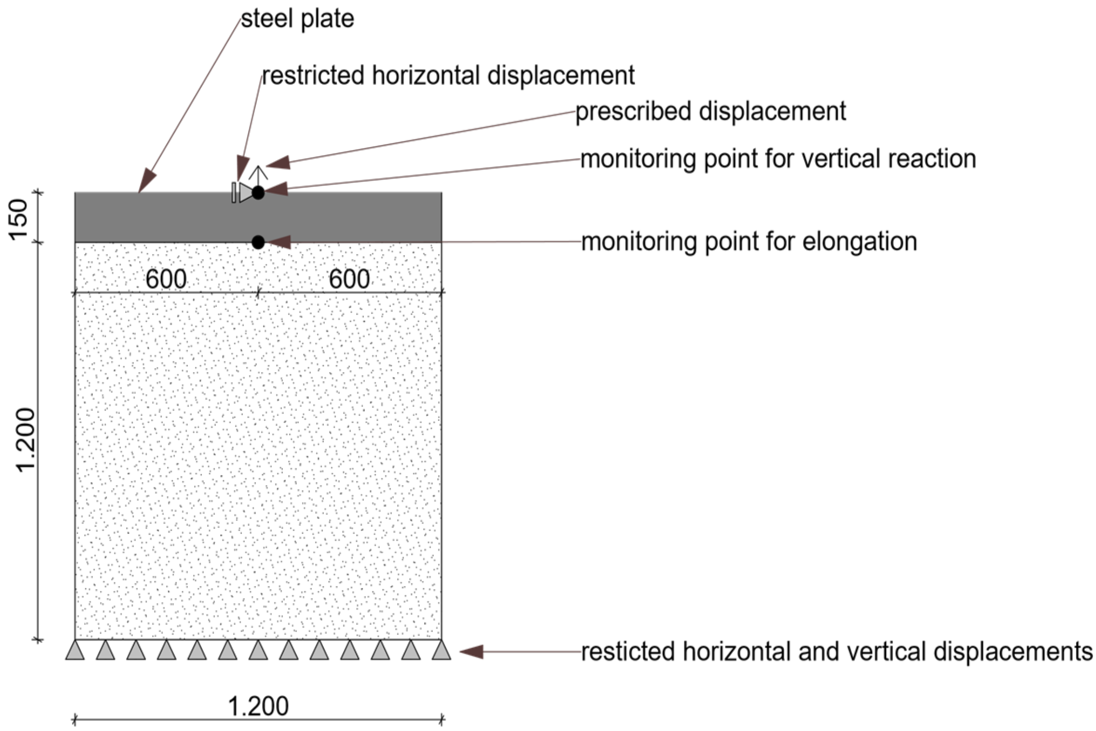


Figure 12: Numerical model arrangement for simulation of a tensile test

In order to determine the tensile strength of the enclosure walls of St. Barbara's church and the tensile strength of the enclosure walls of St. Anna's church, two tensile test simulations were performed. Similar to the compression test simulation, two wall specimens of reduced size were considered. One of the specimens represented the enclosure walls of St. Barbara's church and the other specimen represented the enclosure walls of St. Anna's church. The dimensions of the panels were the same that were used during the compression test simulation.

The numerical model arrangement and the boundary conditions that were considered during the simulation of the tensile test, can be seen in Figure 12. The reaction was monitored at the supports too. The horizontal and vertical displacements at the bottom are restricted. At the top, it is placed a steel plate, which is fixed on the upper surface of the masonry wall, so that during the tensile test, the stone masonry panel is pulled by the steel element and stretched. At the point where the tensile force is applied, the horizontal displacement is restricted and a prescribed vertical displacement is applied.

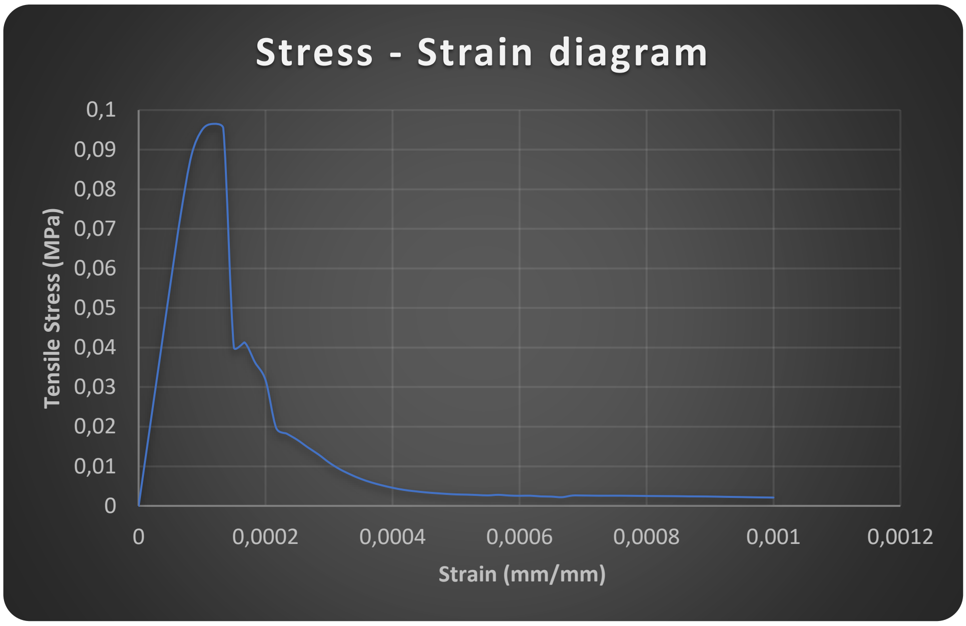
- **St. Anna's church - Enclosure walls**

During the in-site investigation of the stone masonry enclosure walls of St. Annas church, it was clearly seen that there were many different masonry components used during the construction of the church. The masonry units consisted of Grey sandstone, basalt and red sandstone. Also, for the assembly of the units, lime mortar was used. The enclosure walls are represented by the micro-model that can be seen in Figure 4. The panel has height 1.2 meters, width 1.2 meters and thickness 1 meter. The steel element at the top of the panel, that is also included in the micro-model has height 15 centimeters, width 1.2 meters and thickness 1 meter. The micro-model consists of 210 macro-elements, whose material properties were assigned according to the properties of the masonry element that they represented.

A multi-step analysis was considered. At each step of the analysis, a prescribed vertical displacement increment was applied. The maximum number of iterations during the analysis have been selected as 50. The increment was considered to be 0.02 mm for all the steps of the analysis. The size of the mesh was chosen to be 1 cm.

Tensile test simulation results

The tensile stress-strain curve that was obtained by the tensile test simulation can be seen in Figure 13. The maximum tensile stress is 0.0965 MPa, according to the graph. The ultimate tensile strength of the stone masonry specimen is taken also as 0.0965 MPa. The maximum tensile stress is achieved on the seventh step of the simulation, where the compressive strain is 0.000116667, the tensile force that is applied at the top of the steel element is 115.8 kN and the displacement is 0.14 millimeters (Figure 33). During the first 3 load steps the stress strain curve remained linear and the modulus of elasticity was estimated as 1.119 GPa (valid for the particular portion of the curve).



Load step	1	2	3	4	5	6	7	8	9	10
Tensile stress [Mpa]	0,01866	0,03732	0,05596	0,07399	0,08867	0,09500	0,09650	0,09533	0,04008	0,04131
strain [mm/mm]	1,67E-05	3,33E-05	5,00E-05	6,67E-05	8,33E-05	1,00E-04	1,17E-04	1,33E-04	1,50E-04	1,67E-04
Load step	11	12	13	14	15	16	17	18	19	20
Tensile stress [Mpa]	0,03625	0,03173	0,01968	0,01823	0,01667	0,01476	0,01298	0,01095	0,00928	0,00796
strain [mm/mm]	1,83E-04	2,00E-04	2,17E-04	2,33E-04	2,50E-04	2,67E-04	2,83E-04	3,00E-04	3,17E-04	3,33E-04
Load step	21	22	23	24	25	26	27	28	29	30
Tensile stress [Mpa]	0,00682	0,00592	0,00519	0,00457	0,00407	0,00373	0,00344	0,00323	0,00305	0,00291
strain [mm/mm]	3,50E-04	3,67E-04	3,83E-04	4,00E-04	4,17E-04	4,33E-04	4,50E-04	4,67E-04	4,83E-04	5,00E-04
Load step	31	32	33	34	35	36	37	38	39	40
Tensile stress [Mpa]	0,00284	0,00275	0,00264	0,00279	0,00261	0,00255	0,00257	0,00240	0,00234	0,00219
strain [mm/mm]	5,17E-04	5,33E-04	5,50E-04	5,67E-04	5,83E-04	6,00E-04	6,17E-04	6,33E-04	6,50E-04	6,67E-04
Load step	41	42	43	44	45	46	47	48	49	50
Tensile stress [Mpa]	0,00261	0,00261	0,00258	0,00257	0,00257	0,00256	0,00253	0,00250	0,00248	0,00246
strain [mm/mm]	6,83E-04	7,00E-04	7,17E-04	7,33E-04	7,50E-04	7,67E-04	7,83E-04	8,00E-04	8,17E-04	8,33E-04
Load step	51	52	53	54	55	56	57	58	59	60
Tensile stress [Mpa]	0,00244	0,00240	0,00238	0,00236	0,00231	0,00227	0,00222	0,00217	0,00213	0,00209
strain [mm/mm]	8,50E-04	8,67E-04	8,83E-04	9,00E-04	9,17E-04	9,33E-04	9,50E-04	9,67E-04	9,83E-04	1,00E-03

Figure 13: Stress – Strain diagram based on tensile test simulation

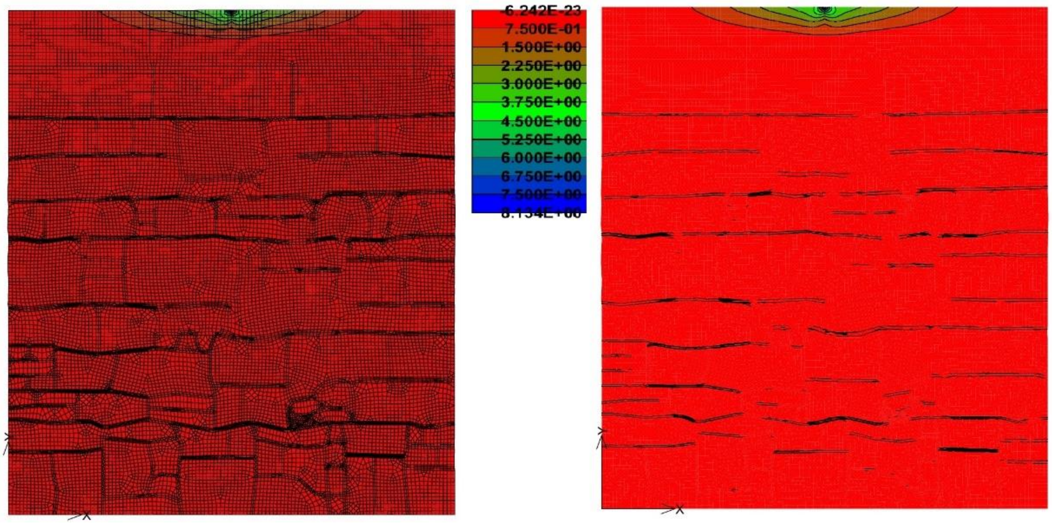
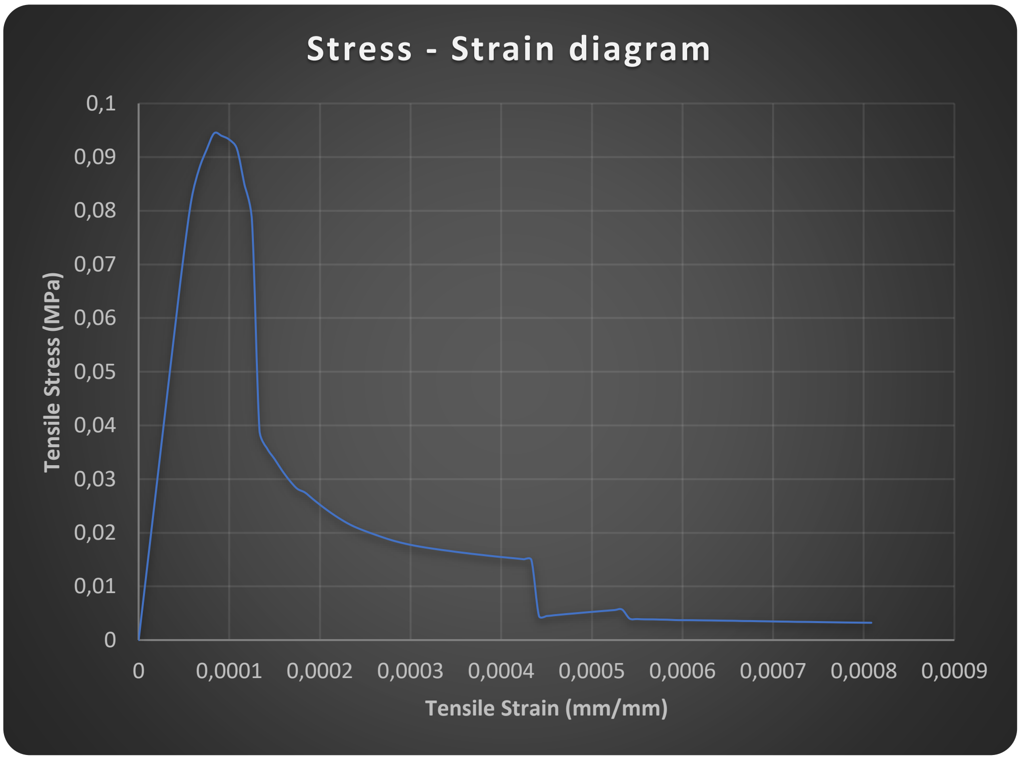


Figure 14: Maximum Principal stress in MPa and crack propagation at step of max. tensile stress (Right: without mesh, undeformed shape / Left: with mesh. deformed shape)

- **St. Barbara's church - Enclosure walls**

In-site investigation of the masonry walls of St. Barbara's church was conducted. One of the tasks of the investigation was to identify the type of the masonry units that were used for the construction of the walls. Another task was to identify if there were some typical stone masonry patterns considered during the construction of the wall or not. This was feasible due to loss of the render in some areas of the wall. The masonry units consisted of Grey sandstone, mudstone and bricks. Also, for the assembly of the units, lime mortar was used. The enclosure walls are represented by the micro-model that can be seen in Figure 7. The panel has height 1.2 meters, width 1.2 meters and thickness 1.2 meters. The steel element at the top of the panel, that is included in the micro-model has height 15 centimeters, width 1.2 meters and thickness 1.2 meters. The material properties of the macro-elements, were assigned according to the properties of the masonry element that they represented. A 97-step analysis was considered. The maximum number of iterations during the analysis have been selected as 50. At each step of the analysis, a prescribed vertical displacement increment was applied. The increment was considered to be 0.01 mm for all the steps of the analysis. The size of the mesh was chosen to be 1 cm.

Tensile test simulation results



Load step	1	2	3	4	5	6	7	8	9	10
Tensile stress [Mpa]	0,0121	0,0242	0,0363	0,0484	0,0605	0,0722	0,0822	0,0878	0,0914	0,0944
Tensile strain [mm/mm]	8,33E-06	1,67E-05	2,50E-05	3,33E-05	4,17E-05	5,00E-05	5,83E-05	6,67E-05	7,50E-05	8,33E-05
Load step	11	12	13	14	15	16	17	18	19	20
Tensile stress [Mpa]	0,0940	0,0933	0,0916	0,0849	0,0782	0,0390	0,0357	0,0337	0,0316	0,0297
Tensile strain [mm/mm]	9,17E-05	1,00E-04	1,08E-04	1,17E-04	1,25E-04	1,33E-04	1,42E-04	1,50E-04	1,58E-04	1,67E-04
Load step	21	22	23	24	25	26	27	28	29	30
Tensile stress [Mpa]	0,0282	0,0275	0,0264	0,0252	0,0242	0,0232	0,0223	0,0215	0,0209	0,0203
Tensile strain [mm/mm]	1,75E-04	1,83E-04	1,92E-04	2,00E-04	2,08E-04	2,17E-04	2,25E-04	2,33E-04	2,42E-04	2,50E-04
Load step	31	32	33	34	35	36	37	38	39	40
Tensile stress [Mpa]	0,0198	0,0193	0,0188	0,0184	0,0181	0,0178	0,0175	0,0172	0,0170	0,0168
Tensile strain [mm/mm]	2,58E-04	2,67E-04	2,75E-04	2,83E-04	2,92E-04	3,00E-04	3,08E-04	3,17E-04	3,25E-04	3,33E-04
Load step	41	42	43	44	45	46	47	48	49	50
Tensile stress [Mpa]	0,0166	0,0164	0,0163	0,0161	0,0159	0,0158	0,0156	0,0155	0,0153	0,0152
Tensile strain [mm/mm]	3,42E-04	3,50E-04	3,58E-04	3,67E-04	3,75E-04	3,83E-04	3,92E-04	4,00E-04	4,08E-04	4,17E-04
Load step	51	52	53	54	55	56	57	58	59	60
Tensile stress [Mpa]	0,0151	0,0149	0,0045	0,0045	0,0046	0,0048	0,0049	0,0050	0,0051	0,0053
Tensile strain [mm/mm]	4,25E-04	4,33E-04	4,42E-04	4,50E-04	4,58E-04	4,67E-04	4,75E-04	4,83E-04	4,92E-04	5,00E-04
Load step	61	62	63	64	65	66	67	68	69	70
Tensile stress [Mpa]	0,0054	0,0055	0,0056	0,0057	0,0040	0,0039	0,0039	0,0039	0,0038	0,0038
Tensile strain [mm/mm]	5,08E-04	5,17E-04	5,25E-04	5,33E-04	5,42E-04	5,50E-04	5,58E-04	5,67E-04	5,75E-04	5,83E-04
Load step	71	72	73	74	75	76	77	78	79	80
Tensile stress [Mpa]	0,0037	0,0037	0,0037	0,0037	0,0037	0,0037	0,0036	0,0036	0,0036	0,0036
Tensile strain [mm/mm]	5,92E-04	6,00E-04	6,08E-04	6,17E-04	6,25E-04	6,33E-04	6,42E-04	6,50E-04	6,58E-04	6,67E-04
Load step	81	82	83	84	85	86	87	88	89	90
Tensile stress [Mpa]	0,0035	0,0035	0,0035	0,0035	0,0035	0,0034	0,0034	0,0034	0,0034	0,0033
Tensile strain [mm/mm]	6,75E-04	6,83E-04	6,92E-04	7,00E-04	7,08E-04	7,17E-04	7,25E-04	7,33E-04	7,42E-04	7,50E-04
Load step	91	92	93	94	95	96	97			
Tensile stress [Mpa]	0,0033	0,0033	0,0033	0,0033	0,0033	0,0032	0,0032			
Tensile strain [mm/mm]	7,58E-04	7,67E-04	7,75E-04	7,83E-04	7,92E-04	8,00E-04	8,08E-04			

Figure 15: Stress – Strain diagram based on tensile test simulation

In Figure 15 we can see the engineering stress-strain curve that was obtained by the simulation of the tensile test. The maximum tensile stress is 0.0944 MPa according to the graph, which is equal to the ultimate tensile strength of the stone masonry specimen. The maximum tensile stress is achieved on the tenth step of the analysis, where the compressive strain is 0,000083325, the tensile force that is applied at the top of the steel element is 136 kN and the displacement is 0.1 millimeter (Figure 39). During the first 5 load steps the engineering stress strain curve remained linear and the modulus of elasticity was estimated as 1.451 GPa (valid for the particular portion of the curve).

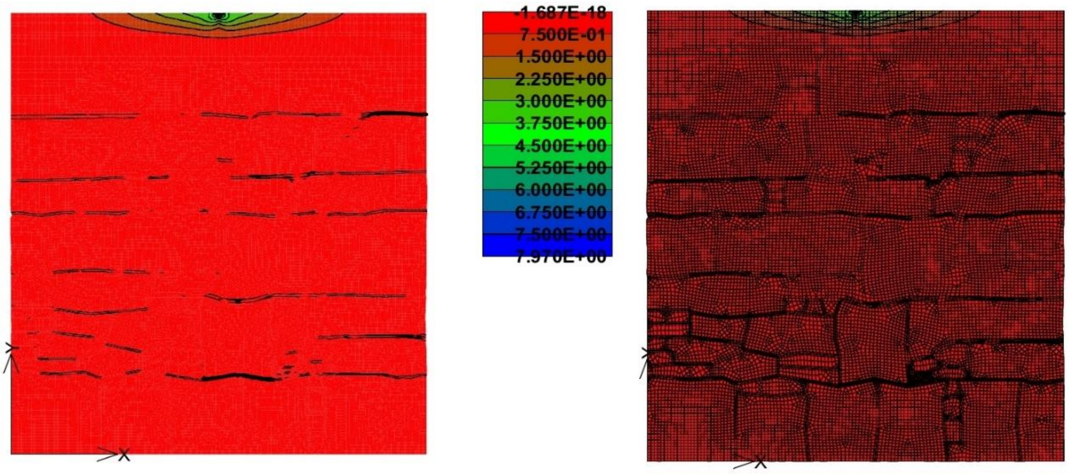


Figure 16: Maximum Principal stress in MPa and crack propagation at step of max. tensile stress (Left: without mesh, undeformed shape / Right: with mesh. deformed shape)

3. Determination of shear strength experimentally

The evaluation of the shear strength of masonry, is feasible by the execution of a diagonal compression test according to ASTM E519-02 standard. During the test, square masonry specimens are used. The test is performed by placing the specimen between two steel elements, which are used for the application of a compressive load on the opposite corners of a masonry sample. The vertical displacements as well as the horizontal displacement are monitored during the test. The minimum size of the specimen is 1.2 meters by 1.2 meters (according to the ASTM standard). The stress due to shear can be calculated as:

$$S_s = \frac{0.707 \times P}{An}$$

where A_n is the net area of the specimen and where P is the diagonal applied load (22).

Determination of shear strength by simulation

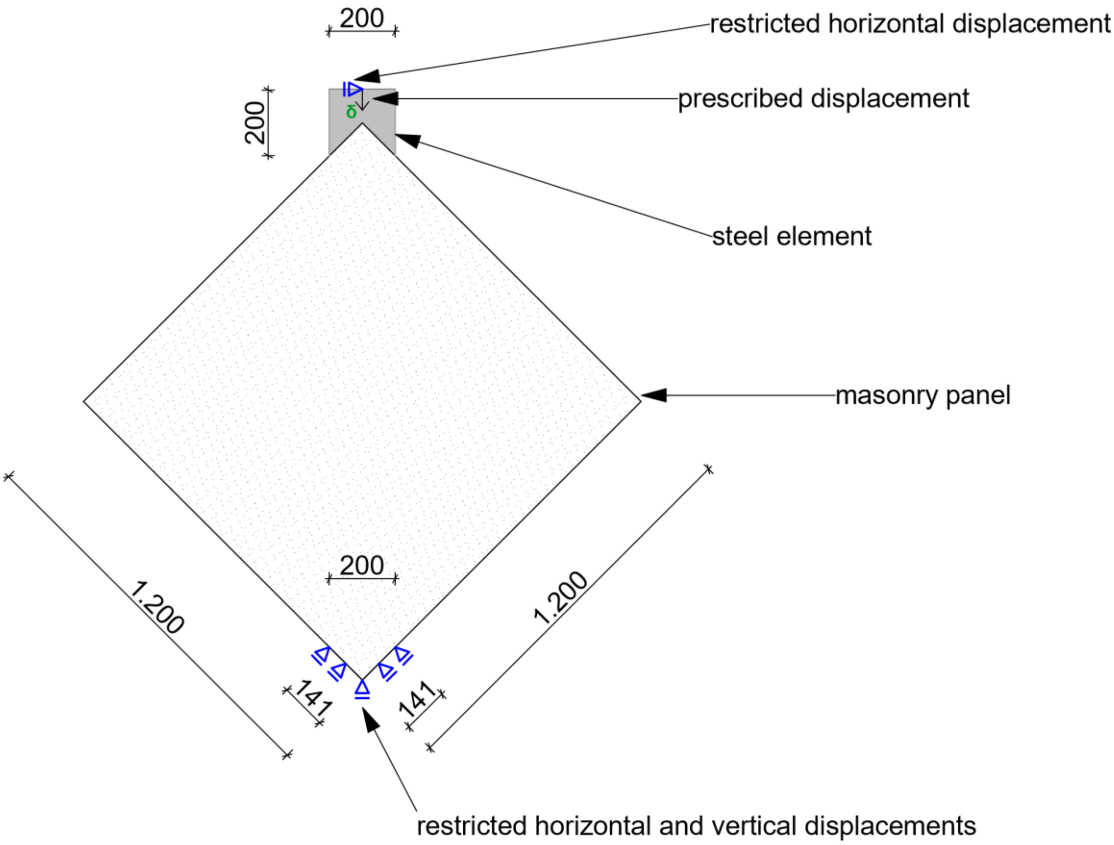
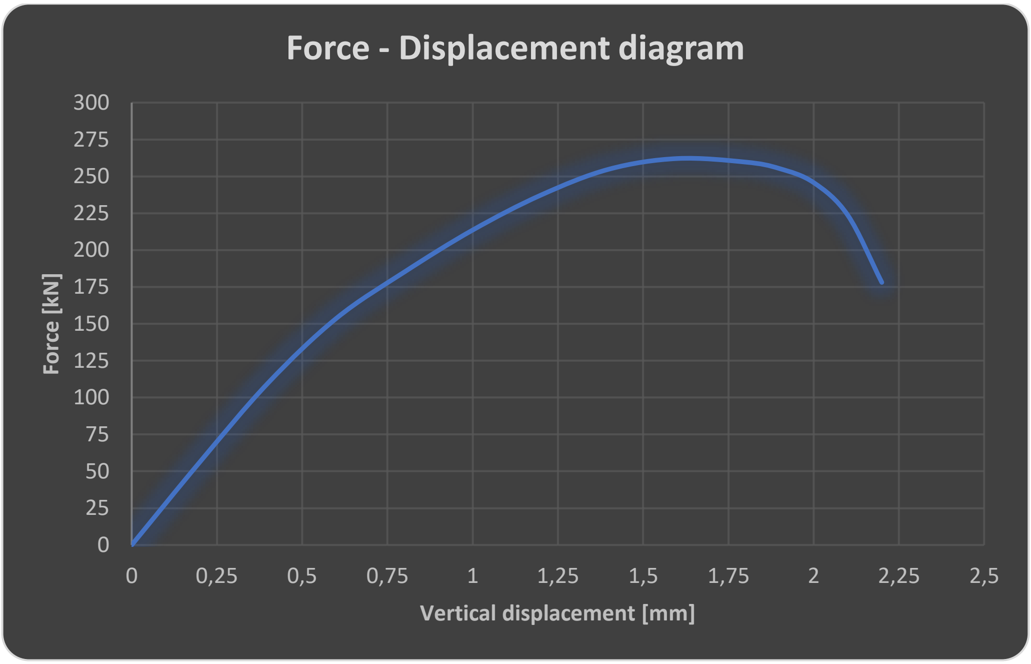


Figure 17: Numerical model arrangement for the simulation of a diagonal compression test

St. Anna's church - Enclosure walls

The arrangement of the numerical model as well as the boundary conditions that were considered during the simulation of the diagonal compression test, can be seen in Figure 17. The thickness of the panel was considered to be 1 meter. The horizontal and vertical displacements at the bottom of the numerical model are restricted as it can be seen in the previous mentioned figure. At the top of the modelled panel, it is placed a steel element (loading shoe made of steel). The loading shoe is used to distribute the load to the masonry panel. At the point where the compressive force is applied, the horizontal displacement is restricted and a prescribed vertical displacement is applied. A monitoring point that recorded the reaction, was positioned at the point of application of the prescribed displacement. The reaction of the line supports was monitored too. In addition to that, monitoring points were applied at the left corner of the masonry panel and at the right corner of the masonry panel. Each of the monitoring points recorded the horizontal displacement of the point of application. Another monitoring point was applied at the upper corner of the masonry panel, which was used to monitor the vertical displacement and to control the prescribed displacement. The size of the mesh was chosen to be 1 centimeter. At each step of the analysis, a prescribed vertical displacement increment was applied. The maximum number of iterations was limited to 50. The increment was considered to be 0.2 mm during the first nine steps of the analysis and 0.1 mm from the tenth load step until the completion of the analysis. In total 13 load steps were considered.

Diagonal compression test simulation results



Load step	1	2	3	4	5
Force [kN]	56,34	109,9	153,7	185,2	213,6
Vertical displacement [mm]	0,2	0,4	0,6	0,8	1
Load step	6	7	8	9	10
Force [kN]	237,3	254,9	262,1	259,7	255,3
Vertical displacement [mm]	1,2	1,4	1,6	1,8	1,9
Load step	11	12	13		
Force [kN]	245,8	223,7	178,1		
Vertical displacement [mm]	2	2,1	2,2		

Figure 18: Force – Displacement diagram based on diagonal compression test simulation

The force – displacement curve that can be seen in Figure 18, was obtained by the simulation of the diagonal compression test. According to the curve, the maximum compressive force that was applied to the masonry panel was 262.1 kN. It was applied on the eighth step of the analysis. After the maximum applied force was identified, the ultimate shear strength was calculated:

$$S_{R.ultimate} = \frac{0.707 \times P_{maximum}}{Anet} = \frac{0.707 \times 262.1 \times 1000}{\left(\frac{1200 + 1200}{2}\right) \times 1000} = 0.154\ MPa$$

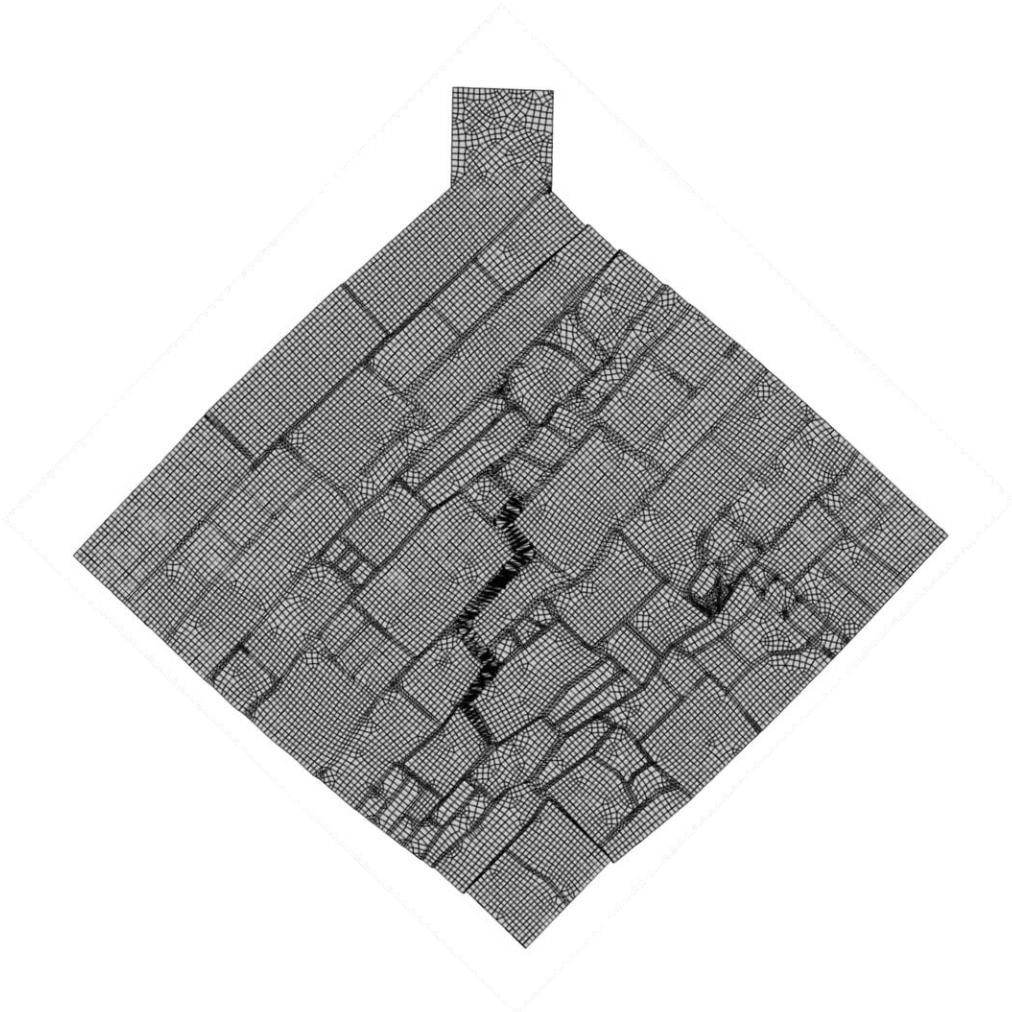


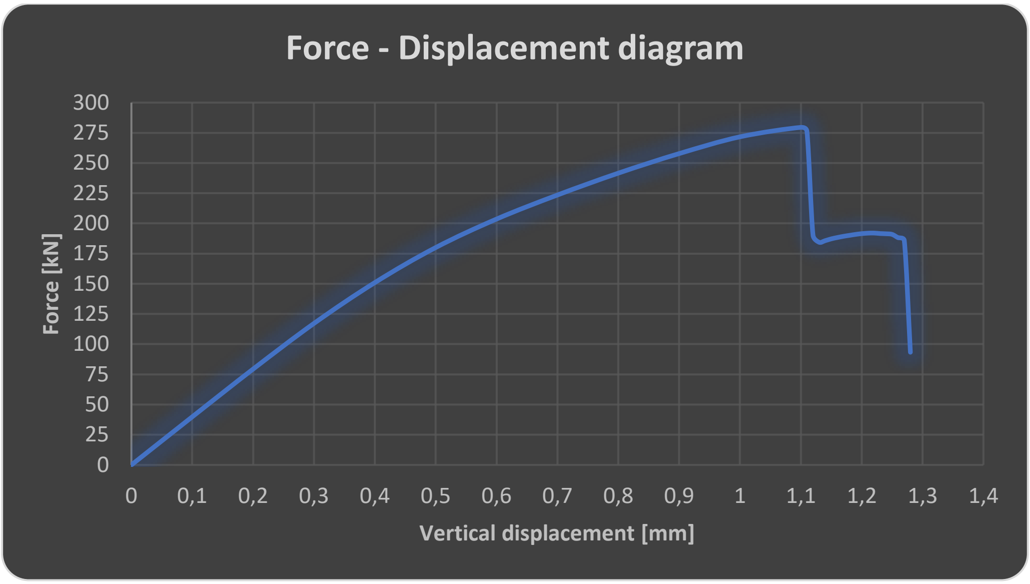
Figure 19: Deformation and crack propagation at step of max. shear stress

St. Barbara’s church - Enclosure walls

The thickness of the masonry specimen that represented the enclosure walls of St. Barbara’s church was chosen as 1.2 meters. The arrangement of the numerical model as well as the boundary conditions that were considered during the simulation of the diagonal compression test, can be seen in Figure 17. In order to monitor the reaction during the analysis, a monitoring point was positioned at the point of application of the prescribed displacement. Also, the reaction was monitored at the supports. Furthermore, monitoring points that recorded the horizontal and vertical displacements were applied, the same way as it was done before.

At each step of the analysis, a prescribed vertical displacement increment was applied. The maximum number of iterations during the analysis have been selected as 50. The increment was considered to be 0.1 mm during the first eleven steps of the analysis and 0.01 mm from the twelfth load step until the completion of the analysis. In total 29 load steps were considered.

Diagonal compression test simulation results



Load step	1	2	3	4	5
Force [kN]	39,86	79,38	117,2	151	180
Vertical displacement [mm]	0,1	0,2	0,3	0,4	0,5
Load step	6	7	8	9	10
Force [kN]	203,6	223,5	241,6	257,8	271,7
Vertical displacement [mm]	0,6	0,7	0,8	0,9	1
Load step	11	12	13	14	15
Force [kN]	279,5	276,3	190,2	184,4	185,7
Vertical displacement [mm]	1,1	1,11	1,12	1,13	1,14
Load step	16	17	18	19	20
Force [kN]	187,2	188,3	189,3	190,1	190,9
Vertical displacement [mm]	1,15	1,16	1,17	1,18	1,19
Load step	21	22	23	24	25
Force [kN]	191,5	191,9	192	191,6	191,4
Vertical displacement [mm]	1,2	1,21	1,22	1,23	1,24
Load step	26	27	28	29	
Force [kN]	190,9	188,1	185,9	93,3	
Vertical displacement [mm]	1,25	1,26	1,27	1,28	

Figure 20: Force – Displacement diagram based on the diagonal compression test simulation

The force – displacement curve that can be seen in Figure 20, was obtained by the simulation of the diagonal compression test. According to the curve, the maximum compressive force that was applied to the masonry panel was 279.5 kN.   It was applied on the eleventh step of the analysis, which is also the load step of the maximum shear stress. After the maximum applied force was identified, the ultimate shear strength was calculated:

$$S_{R.ultimate} = \frac{0.707 \times P_{maximum}}{Anet} = \frac{0.707 \times 279.5 \times 1000}{\left(\frac{1200 + 1200}{2}\right) \times 1200} = 0.137\ MPa$$

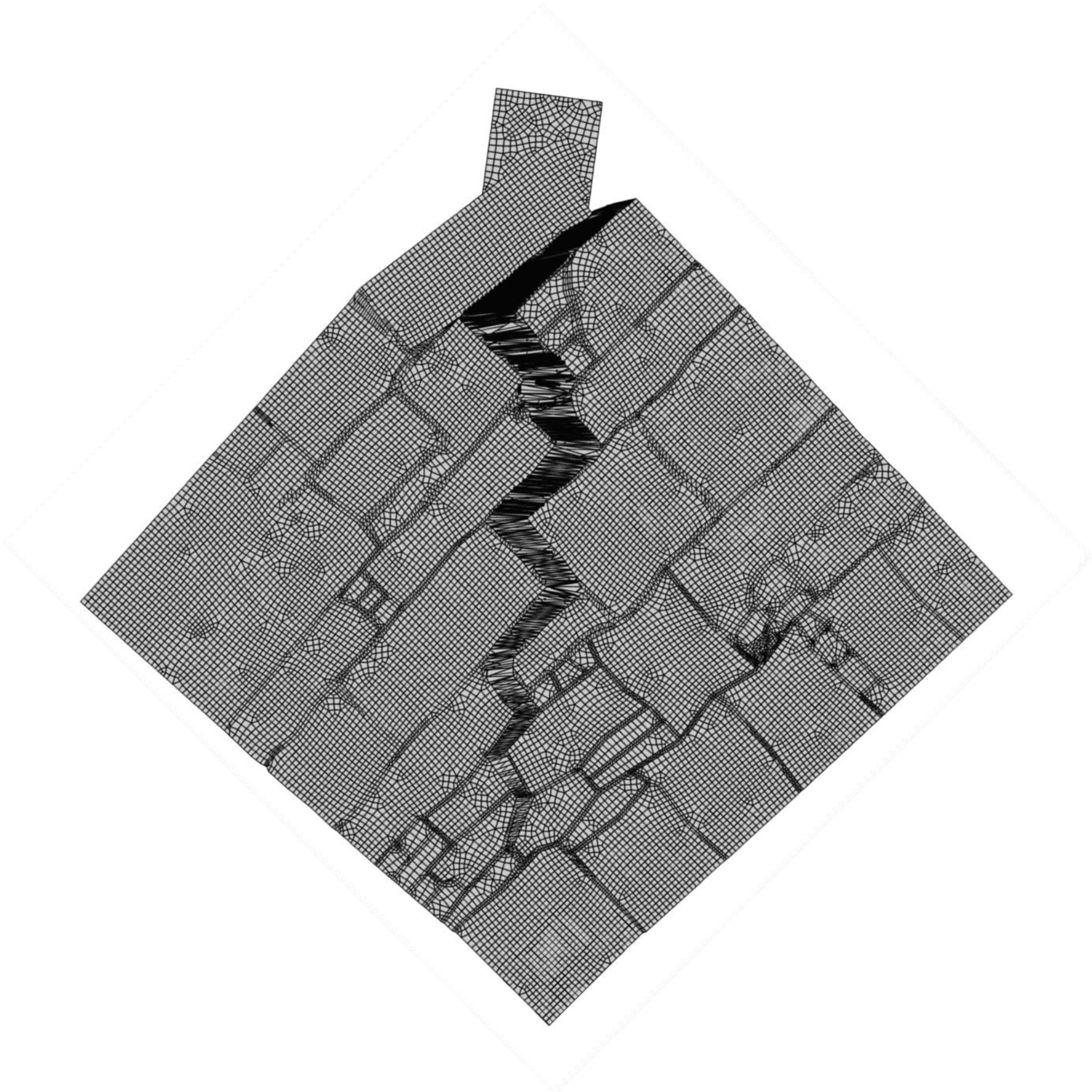


Figure 21: Deformation and crack propagation at last step of analysis (Displacement multiplier K=3.0E+01)

4. Strength versus Stress

St. Barbara's enclosure walls

During the stress analysis, the self-weight of the masonry was considered to be approximately 20.5 $\frac{kN}{m^3}$. The average thickness of the stone masonry enclosure wall was taken as 1.2 meters. The floor plan of the church is 31.7 meters long and 20.5 meters wide, including the thickness of the wall, whose height is about 15 meters. The total dead load of the roofing system was estimated as 1.5 $\frac{kN}{m^2}$ and included all the structural elements and components of the roof. According to figure map, the Broumov region belongs in the fourth area and the characteristic value of the snow load is 2.25 $\frac{kN}{m^2}$. The analysis was performed on a two-dimensional model of the wall, with height 15 meters and length 31.7 meters. The maximum number of iterations during the analysis have been selected as 50. The model represents the projection of the enclosure wall in the X-Y plane, so that we can take advantage of the symmetry of the structure. According to the boundary conditions, the horizontal and vertical displacements are restricted at the bottom of the wall.

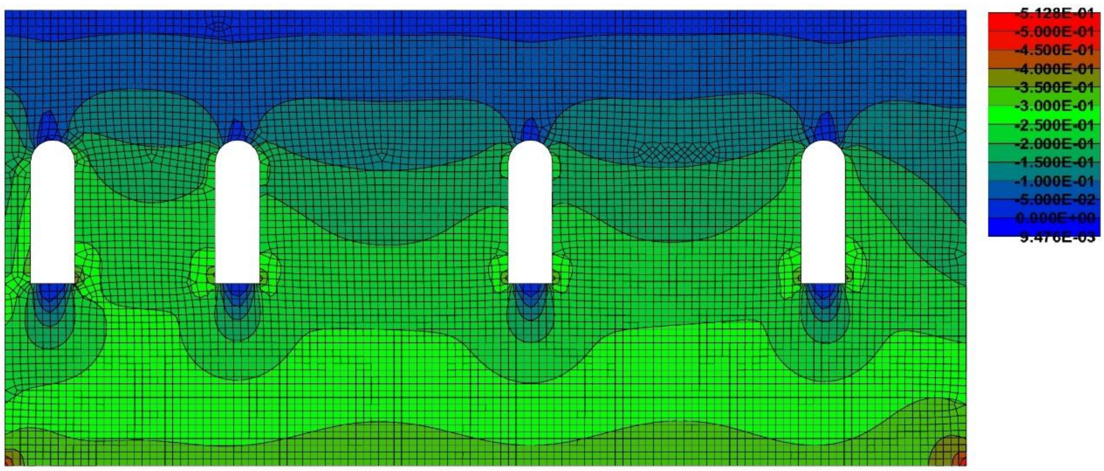


Figure 22: Minimum principal stress in MPa according to the current loading conditions

As it can be seen in Figure 22, the maximum compressive stress of the wall due to the current loading situation is about 0.5128 MPa, which is about 28.5% of the ultimate compressive strength of the stone masonry specimen (1.7965 MPa), as it was determined by the compression test simulation. There is a concentration of stress at the lower corners of the wall, which is the location of the maximum compression stress level.

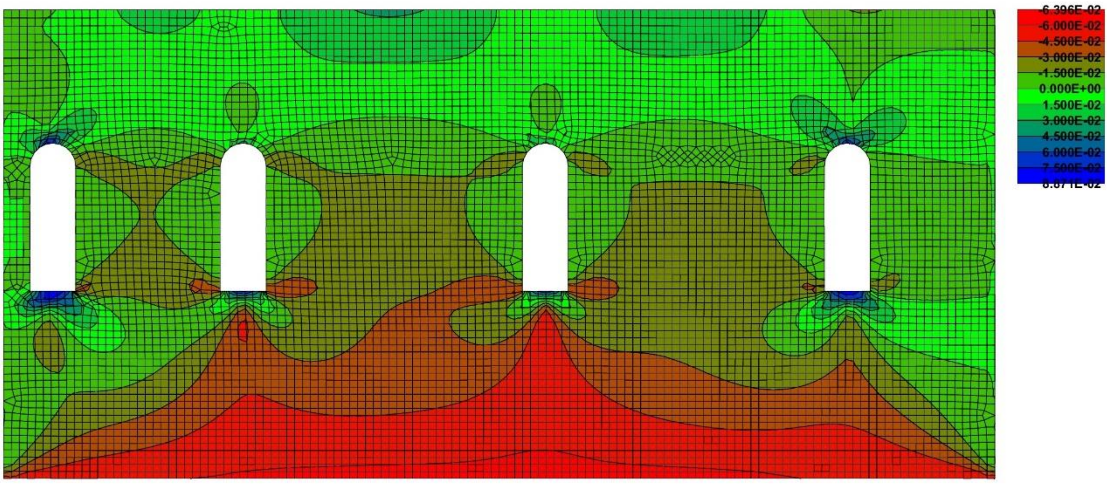


Figure 23: Maximum principal stress in MPa according to the current loading conditions

According to Figure 23, the maximum tensile stress of the wall due to the current loading situation is about 0.08871 MPa, which is about 93.9% of the ultimate tensile strength of the stone masonry specimen (0.0944 MPa), as it was estimated during the tensile test simulation. There is concentration of tensile stresses at the bottom of the openings and at the top of them.

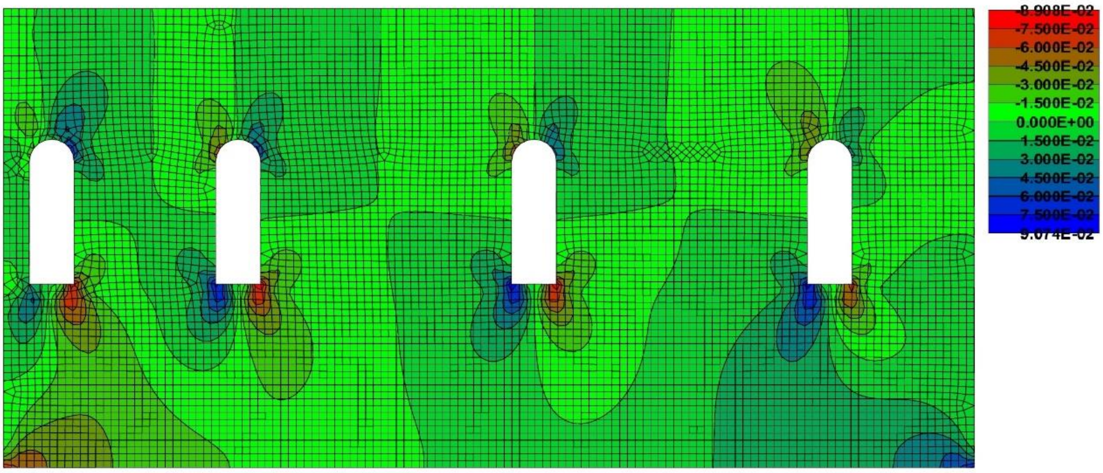


Figure 24: Shear stress in MPa according to the current loading conditions

Based on figure 11, the maximum shear stress acting on the wall is about 0.09074 MPa, which is about 66.2% of the ultimate shear strength of the stone masonry specimen (0.137 MPa), as it was estimated by the diagonal compression test simulation. We can clearly see at the previously noted figure, that there is shear stress concentration at the lower corners of the openings, in addition there is concentration at the upper parts of the openings and at the lower corners of the wall.

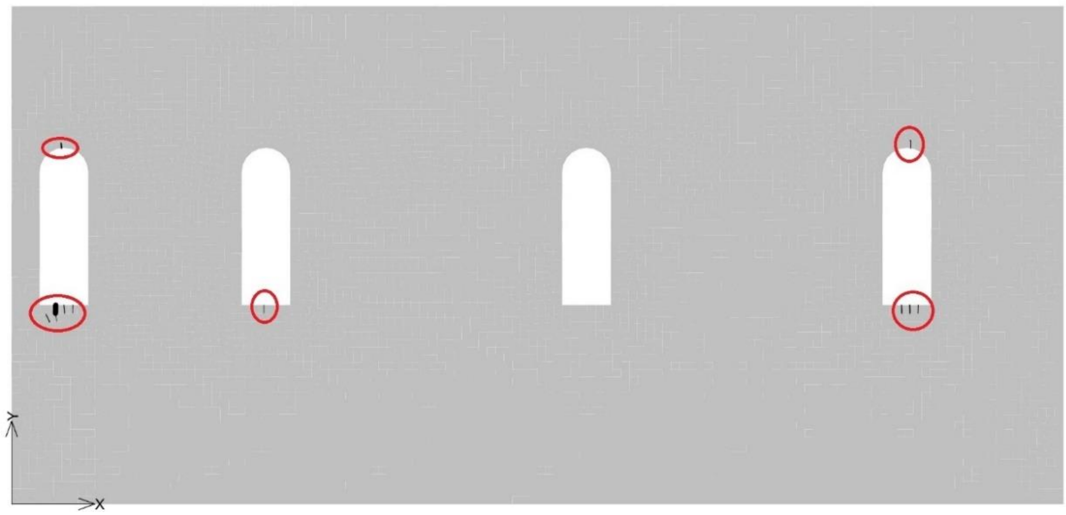


Figure 25: Cracking due to the current loading conditions

In Figure 24: the location of cracks due to the current load can be observed. The minimum width of the cracks is 1.551×10^{-6} $meters$ and the maximum width of the cracks is 6.080×10^{-5} $meters$.

St. Anna's enclosure walls

The self-weight of the masonry was considered to be 21 $\frac{kN}{m^3}$. The average thickness of the stone masonry enclosure wall was taken as 1 meter. The floor plan of the church is 46.5 meters long and 24 meters wide, including the thickness of the wall, whose height is about 17 meters. The total dead load of the roofing system was estimated as 1.7 $\frac{kN}{m^2}$. According to figure map, the Broumov region belongs in the fourth area and the characteristic value of the snow load is 2.25 $\frac{kN}{m^2}$. The analysis was performed on a two-dimensional model of the wall, with height 17 meters and length 46.5 meters. The maximum number of iterations during the analysis have been selected as 50. The model represents the projection of the enclosure wall in the X-Y plane, so that we can take advantage of the symmetry of the structure. According to the boundary conditions, the horizontal and vertical displacements are restricted at the bottom of the wall.

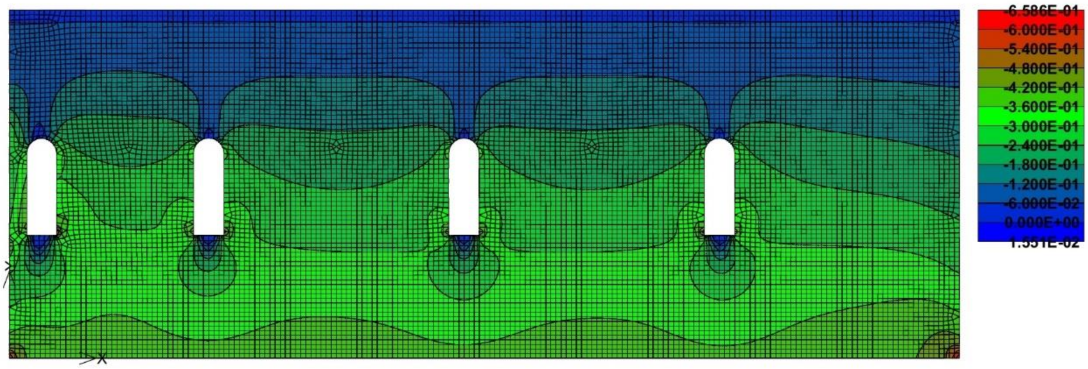


Figure 26: Minimum principal stress in MPa according to the current loading conditions

As it can be seen in Figure25, the maximum compressive stress of the wall due to the current loading situation is about 0.6586 MPa, which is about 37.2% of the ultimate compressive strength of the stone masonry specimen (1.769 MPa), as it was determined by the compression test simulation. There is a concentration of stress at the lower corners of the wall, which is the location of the maximum compression stress level.

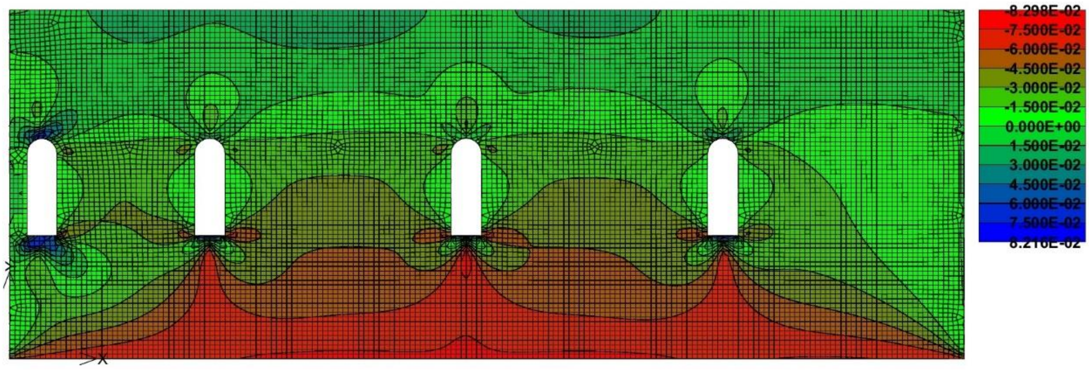


Figure 27: Maximum principal stress in MPa according to the current loading conditions

According to figure 26, the maximum tensile stress of the wall due to the current loading conditions is about 0.0821 MPa, which is about 84.9% of the ultimate tensile strength of the stone masonry specimen (0.0965 MPa), as it was estimated during the tensile test simulation. There is concentration of tensile stresses at the bottom of some of the openings as well as at the top.

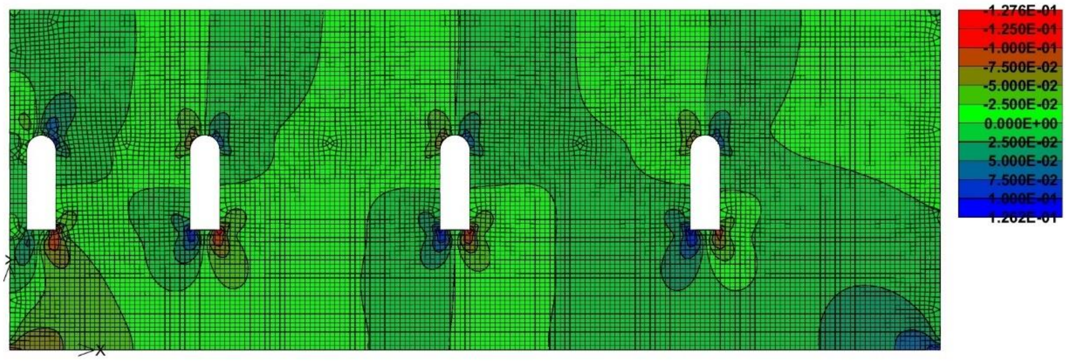


Figure 28: Shear stress in MPa according to the current loading conditions

The distribution of the shear stress based on the current loading conditions, is seen in figure 27, the maximum shear stress acting on the wall is about 0.127 MPa, which is about 82.4% of the ultimate shear strength of the stone masonry specimen (0.154 MPa), as it was estimated by the diagonal compression test simulation. We can clearly see that there is shear stress concentration at the lower corners of the openings. In addition, there is concentration at the upper parts of the openings and at the lower corners of the wall.

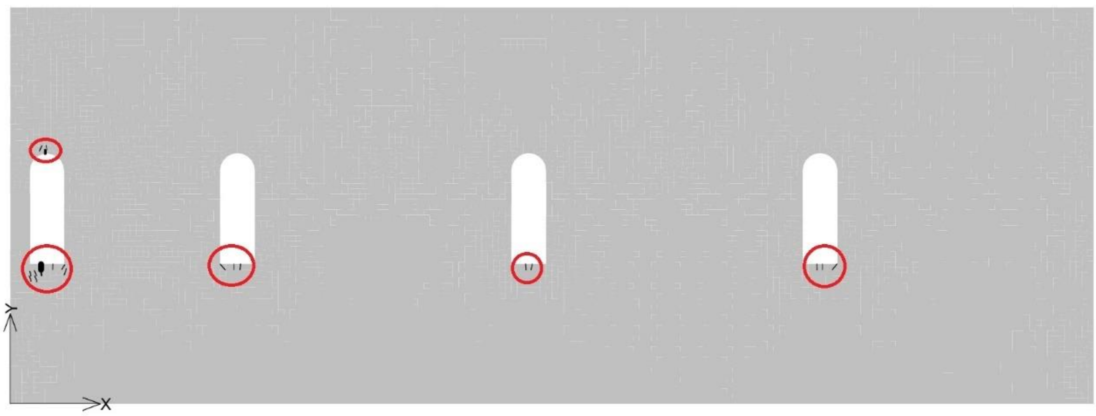
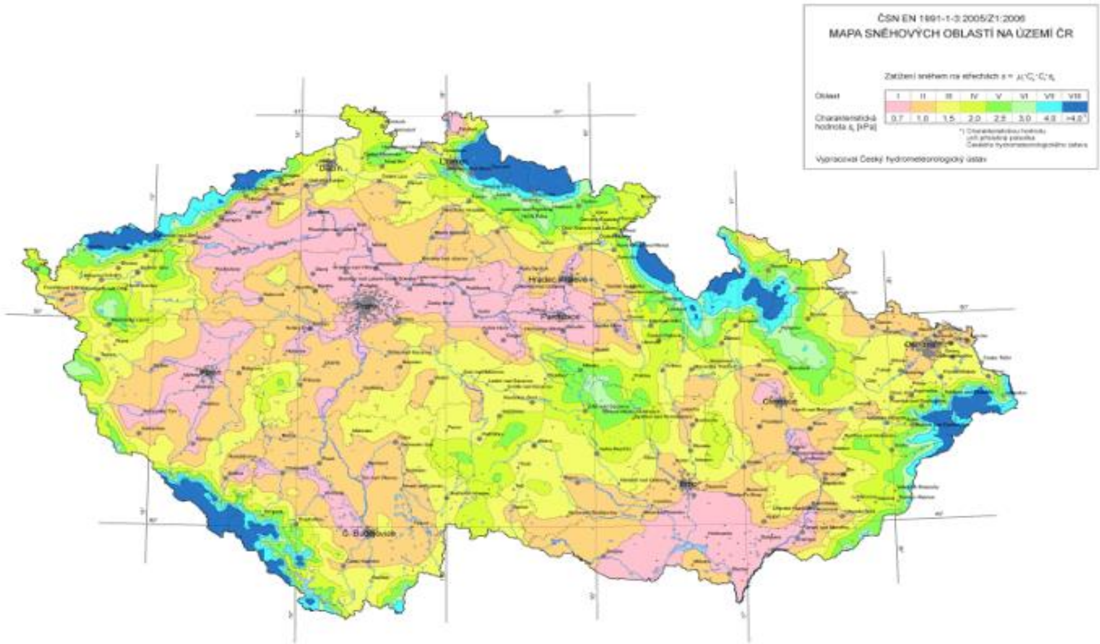


Figure 29: Cracking due to the current loading conditions

In Figure 29: the location of cracks due to the current load can be observed. The minimum width of the cracks is 3.178×10^{-7} $meters$ and the maximum width of the cracks is 1.798×4 $meters$.



Picture 35: Snow map of Czech republic

Source: https://stavba.tzb-info.cz/tabulky-a-vypocty/143-mapa-snehovych-oblasti-na-uzemi-ceske-republiky

ČSN EN 1991-1-3,Eurocode 1: Actions on structures Part 1-3 General actions – Snow Loads, 2005

IX. Conclusion

The Broumov Group of Churches has a unique position in the Baroque architecture, as they were made in a very short period of time in the Broumov region and they were designed by the same architects (Dientzenhofers and Allio).

In my work I have focused on two of the nine churches. Both of the churches are in bad condition. The main reason for that is the lack of maintenance, however, in the case of St. Anna the insufficient gutter-system plays an important role too (it was not considered in this work the effect of it). The Broumov region is very rich in sandstone, this is the reason why sandstone (red, green, gray, etc...) is the main material that was used during the construction of both churches. Because of the loss of render, the stones that were used for the construction, are exposed to weathering. Of course an important role on the damage play the freeze-thaw cycles too.

All these combined together, have caused damage to the walls of the churches. In order to be able to determine the severity of the damage and the danger of structural failure, I have performed series of tests, so that I can determine the compressive, the tensile and the shear strength of the walls. Because of the masonry properties (is very heterogenous), I created micromodels. The tests were performed by the software ATENA Engineering 2D of Červenka Consulting.

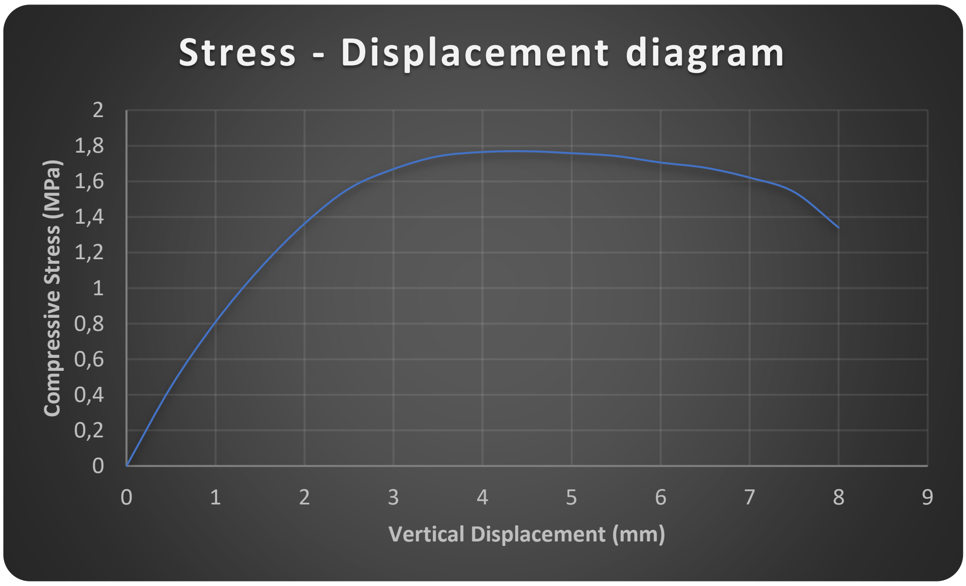
The obtained results were demonstrated by graphs, tables and pictures of the micromodels. According of the obtained results, I can conclude that the investigated churches (St. Anna and St. Barbara), at the time of investigation are not in danger of serious structural failure.

In order to be able to determine, the exact reason of the wall damages, further investigation is necessary. It should take also into consideretion the effect of the missing gatter system. Also proper reparation of the damages would be necessary. Until the reconstruction of the churches is realized, a watchfull-waiting approach can be applied.

X Appendix

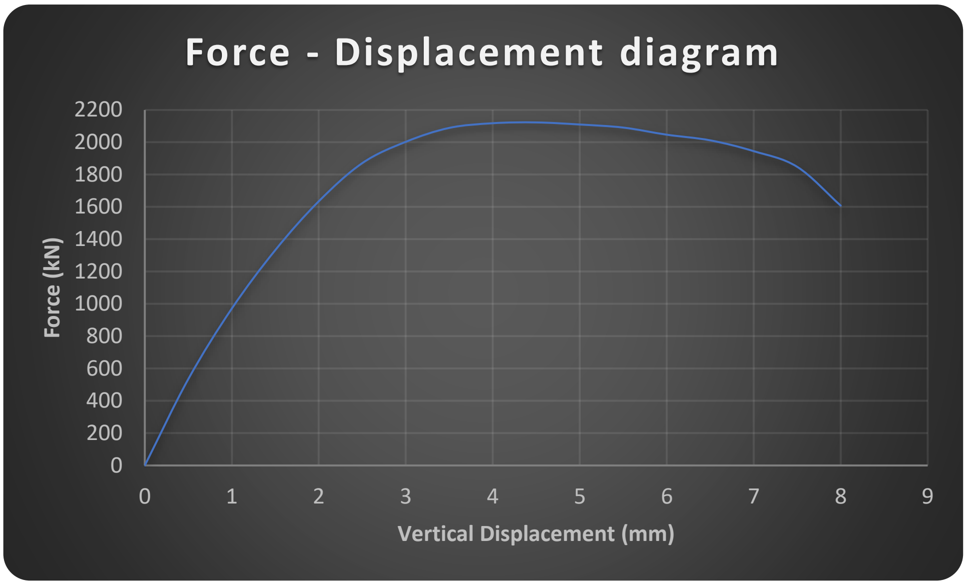
1. Compression test simulation

St. Anna's enclosure walls



Load step	1	2	3	4	5	6	7	8
compressive stress [Mpa]	0,446	0,809	1,111	1,363	1,559	1,669	1,741	1,765
Vertical displacement [mm]	0,5	1	1,5	2	2,5	3	3,5	4
Load step	9	10	11	12	13	14	15	16
compressive stress [Mpa]	1,769	1,758	1,743	1,706	1,677	1,621	1,540	1,340
Vertical displacement [mm]	4,5	5	5,5	6	6,5	7	7,5	8

Figure. 30: Stress - displacement diagram based on compression test simulation



Load step	1	2	3	4	5	6	7	8
Force [kN]	534,8	971	1333	1635	1871	2003	2089	2118
Vertical displacement [mm]	0,5	1	1,5	2	2,5	3	3,5	4
Load step	9	10	11	12	13	14	15	16
Force [kN]	2123	2110	2091	2047	2012	1945	1848	1608
Vertical displacement [mm]	4,5	5	5,5	6	6,5	7	7,5	8

Figure 31: Force - Displacement diagram based on compression test simulation

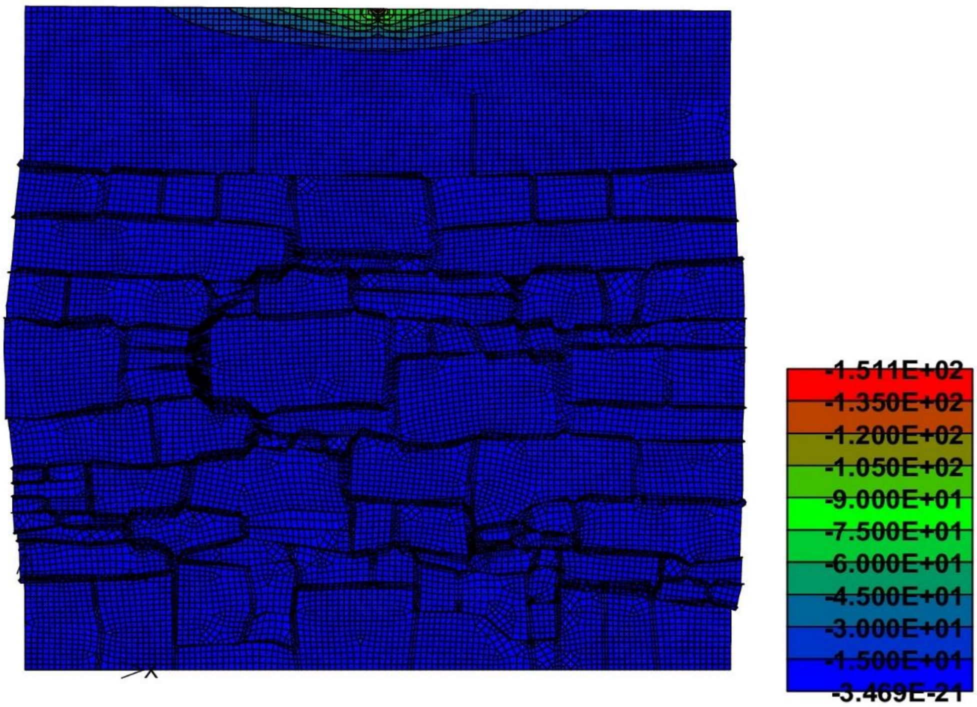
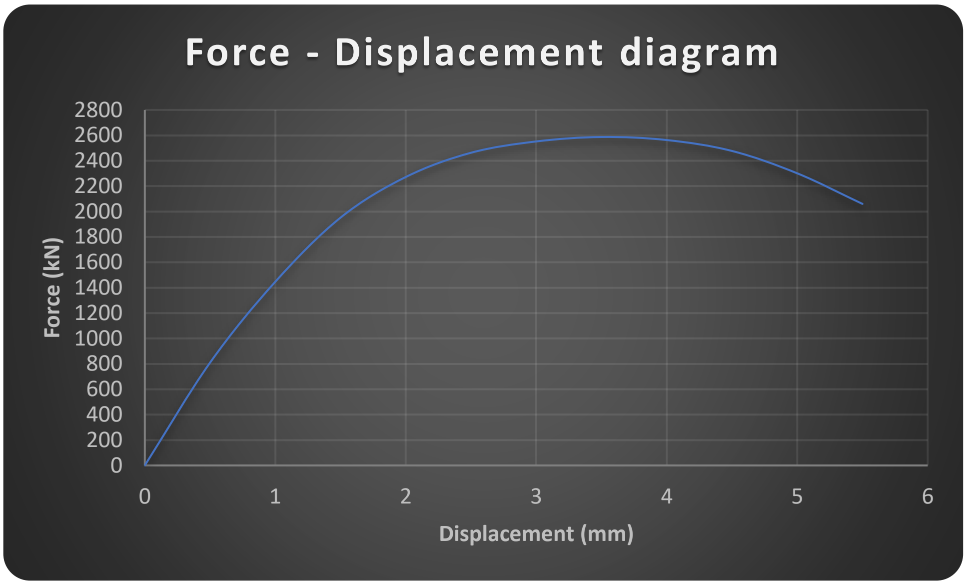


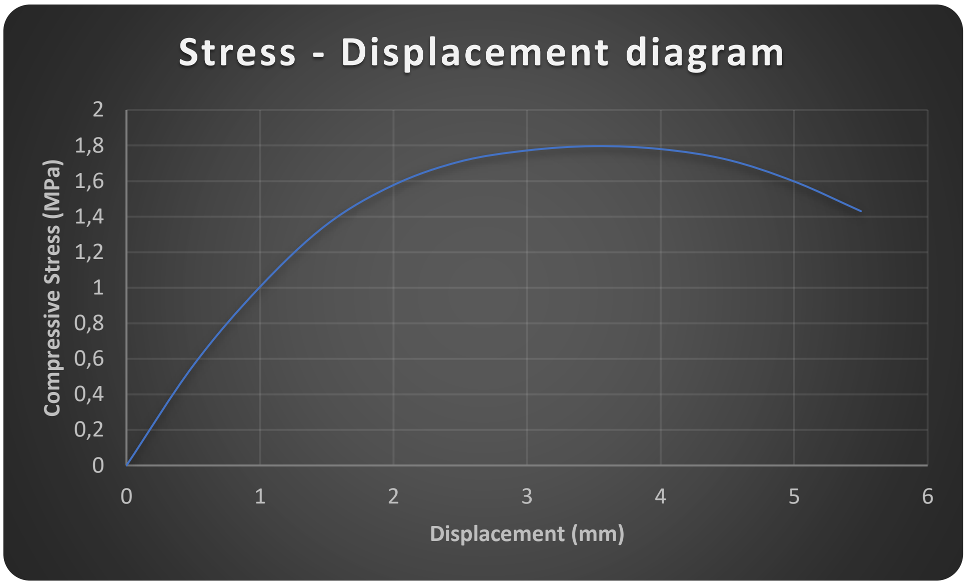
Figure 32: Minimum Principal stress and crack propagation (step of max. compressive stress)

St. Barbara's enclosure walls



Load step	1	2	3	4	5	6
Force [kN]	809,7	1447	1952	2272	2463	2552
Displacement [mm]	0,5	1	1,5	2	2,5	3
Load step	7	8	9	10	11	
Force [kN]	2587	2563	2477	2301	2060	
Displacement [mm]	3,5	4	4,5	5	5,5	

Figure 33: Force - Displacement diagram based on compression test simulation



Load step	1	2	3	4	5	6
Compressive stress [Mpa]	0,5623	1,0049	1,3556	1,5778	1,7104	1,7722
Displacement [mm]	0,5	1	1,5	2	2,5	3
Load step	7	8	9	10	11	
Compressive stress [Mpa]	1,7965	1,7799	1,7201	1,5979	1,4306	
Displacement [mm]	3,5	4	4,5	5	5,5	

Figure 34: Stress - displacement diagram based on compression test simulation

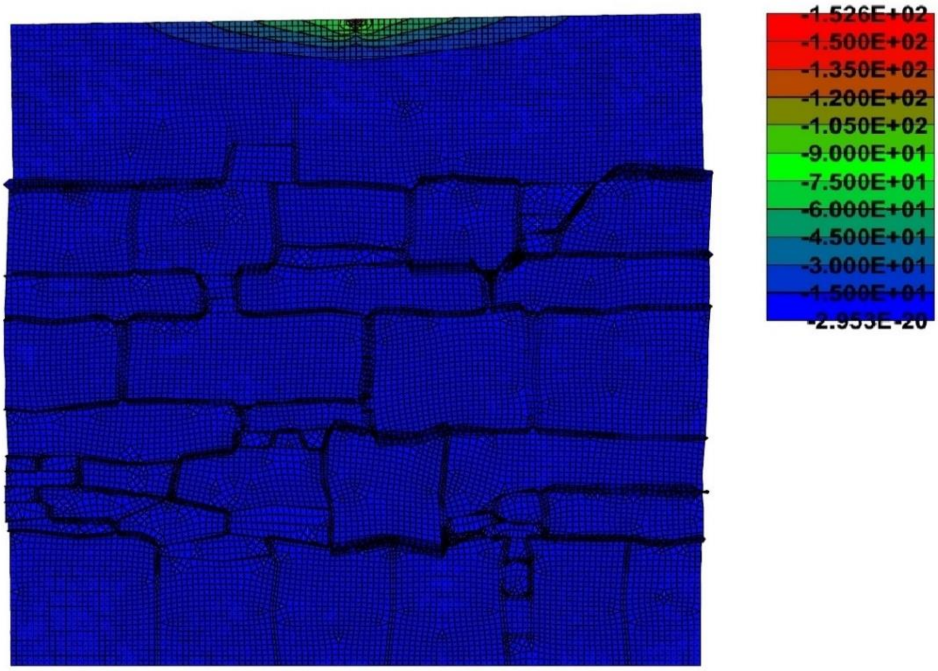
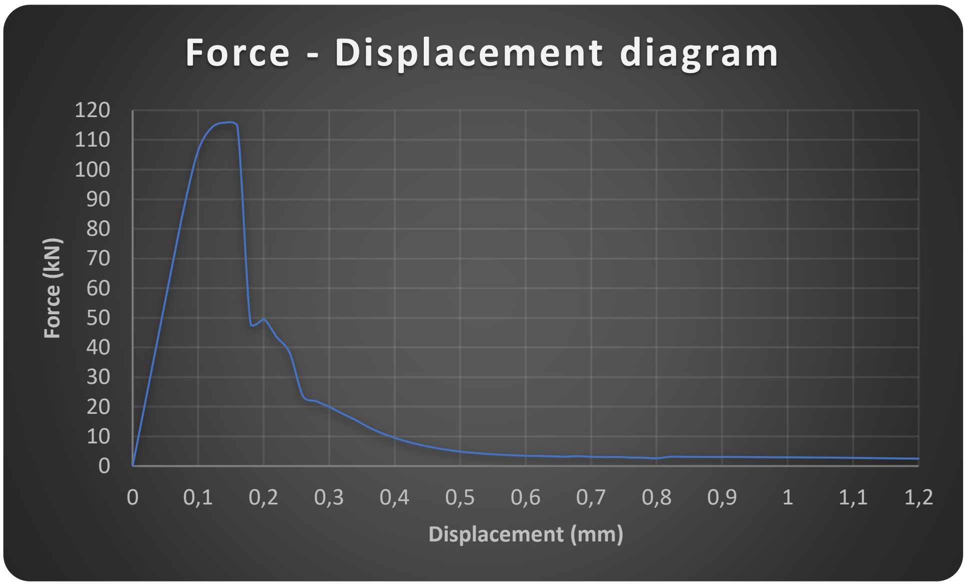


Figure 35: Minimum Principal stress and crack propagation (step of max. compressive stress)

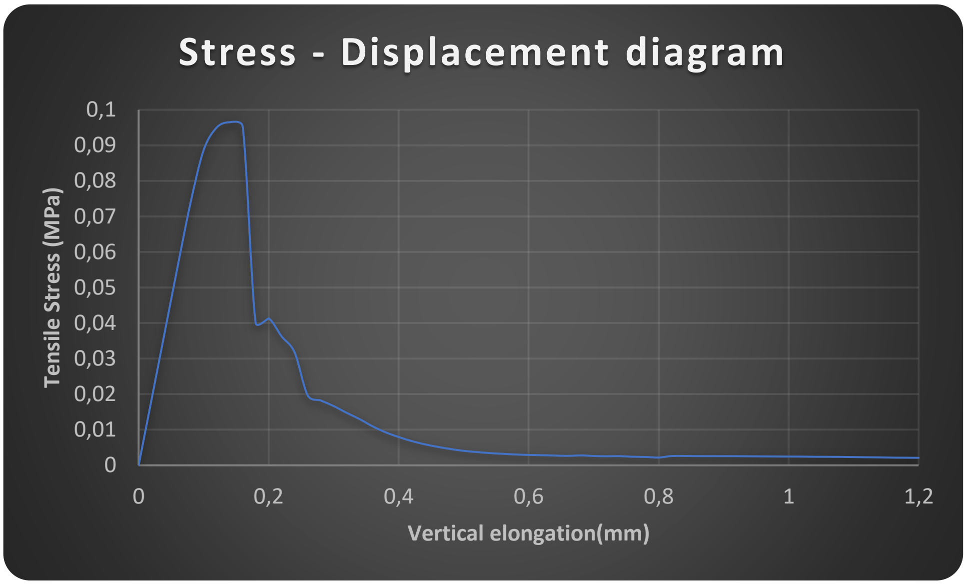
2. Tensile test simulation

St. Anna’s church - Enclosure walls



Load step	1	2	3	4	5	6	7	8	9	10
Force [kN]	22,39	44,78	67,15	88,79	106,4	114	115,8	114,4	48,09	49,57
Displacement [mm]	0,02	0,04	0,06	0,08	0,1	0,12	0,14	0,16	0,18	0,2
Load step	11	12	13	14	15	16	17	18	19	20
Force [kN]	43,5	38,08	23,62	21,88	20	17,71	15,58	13,14	11,13	9,548
Displacement [mm]	0,22	0,24	0,26	0,28	0,3	0,32	0,34	0,36	0,38	0,4
Load step	21	22	23	24	25	26	27	28	29	30
Force [kN]	8,187	7,101	6,23	5,488	4,885	4,472	4,131	3,872	3,654	3,486
Displacement [mm]	0,42	0,44	0,46	0,48	0,5	0,52	0,54	0,56	0,58	0,6
Load step	31	32	33	34	35	36	37	38	39	40
Force [kN]	3,409	3,301	3,172	3,347	3,136	3,058	3,089	2,882	2,813	2,628
Displacement [mm]	0,62	0,64	0,66	0,68	0,7	0,72	0,74	0,76	0,78	0,8
Load step	41	42	43	44	45	46	47	48	49	50
Force [kN]	3,128	3,135	3,093	3,08	3,088	3,074	3,041	3,001	2,972	2,948
Displacement [mm]	0,82	0,84	0,86	0,88	0,9	0,92	0,94	0,96	0,98	1
Load step	51	52	53	54	55	56	57	58	59	60
Force [kN]	2,928	2,881	2,86	2,828	2,771	2,718	2,667	2,6	2,558	2,513
Displacement [mm]	1,02	1,04	1,06	1,08	1,1	1,12	1,14	1,16	1,18	1,2

Figure 36: Force - Displacement curve based on tensile test simulation



Load step	1	2	3	4	5	6	7	8	9	10
Tensile stress [Mpa]	0,018658	0,037317	0,055958	0,073992	0,088667	0,095	0,0965	0,095333	0,040075	0,041308
Displacement [mm]	0,02	0,04	0,06	0,08	0,1	0,12	0,14	0,16	0,18	0,2
Load step	11	12	13	14	15	16	17	18	19	20
Tensile stress [Mpa]	0,03625	0,031733	0,019683	0,018233	0,016667	0,014758	0,012983	0,01095	0,009275	0,007957
Displacement [mm]	0,22	0,24	0,26	0,28	0,3	0,32	0,34	0,36	0,38	0,4
Load step	21	22	23	24	25	26	27	28	29	30
Tensile stress [Mpa]	0,006823	0,005918	0,005192	0,004573	0,004071	0,003727	0,003443	0,003227	0,003045	0,002905
Displacement [mm]	0,22	0,24	0,26	0,28	0,3	0,32	0,34	0,36	0,38	0,4
Load step	31	32	33	34	35	36	37	38	39	40
Tensile stress [Mpa]	0,002841	0,002751	0,002643	0,002789	0,002613	0,002548	0,002574	0,002402	0,002344	0,00219
Displacement [mm]	0,62	0,64	0,66	0,68	0,7	0,72	0,74	0,76	0,78	0,8
Load step	41	42	43	44	45	46	47	48	49	50
Tensile stress [Mpa]	0,002607	0,002613	0,002578	0,002567	0,002573	0,002562	0,002534	0,002501	0,002477	0,002457
Displacement [mm]	0,82	0,84	0,86	0,88	0,9	0,92	0,94	0,96	0,98	1
Load step	51	52	53	54	55	56	57	58	59	60
Tensile stress [Mpa]	0,00244	0,002401	0,002383	0,002357	0,002309	0,002265	0,002223	0,002167	0,002132	0,002094
Displacement [mm]	1,02	1,04	1,06	1,08	1,1	1,12	1,14	1,16	1,18	1,2

Figure 37: Stress - Displacement curve based on tensile test simulation

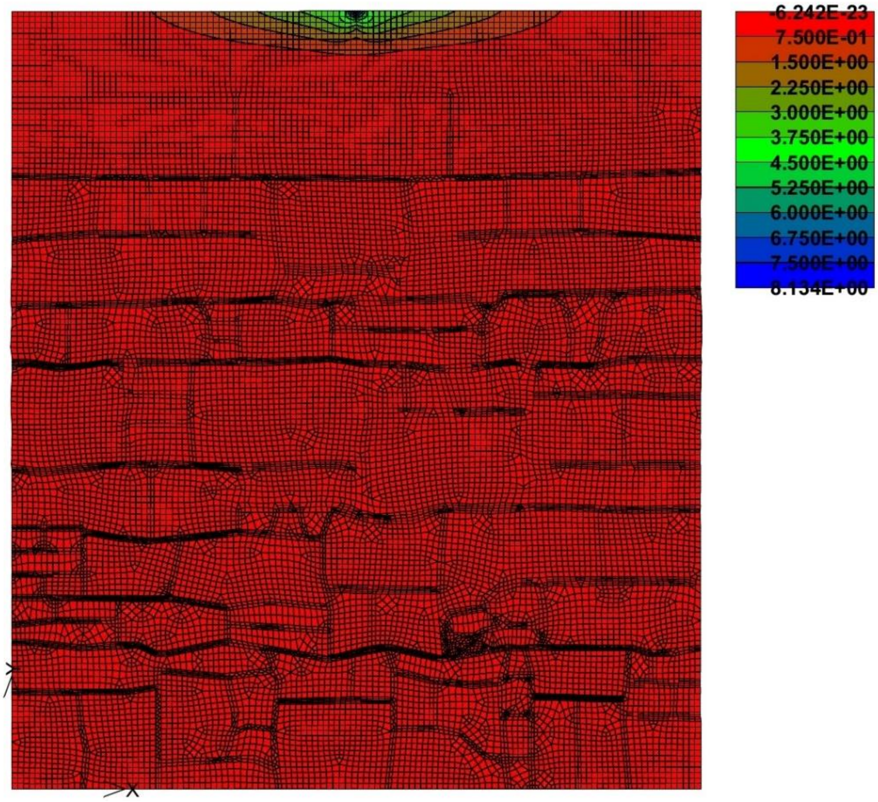
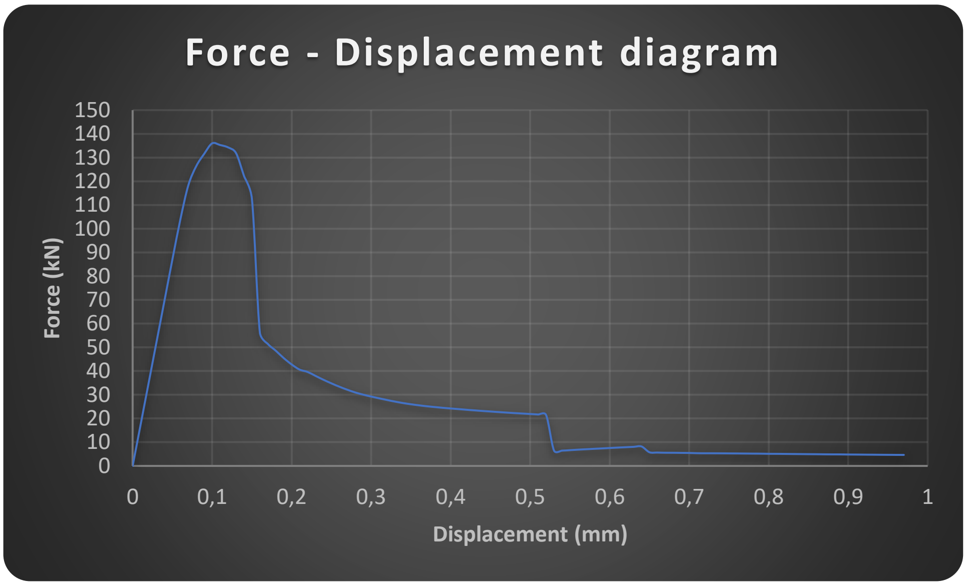


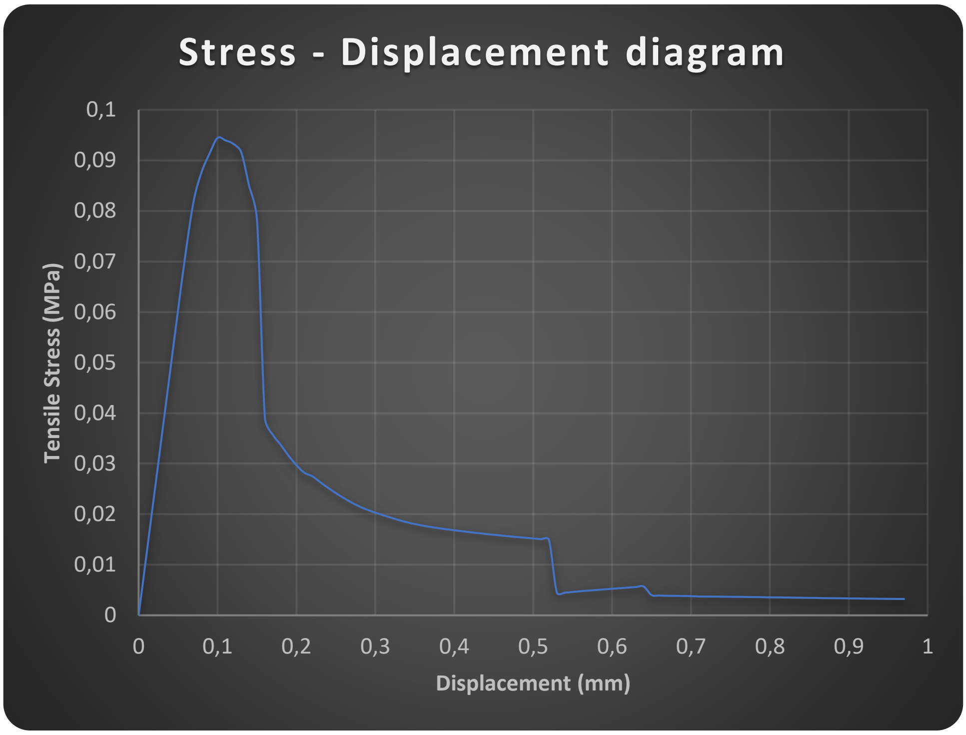
Figure 38: Maximum Principal stress and crack propagation (step of max. tensile stress)

St. Barbara’s church - Enclosure walls



Load step	1	2	3	4	5	6	7	8	9	10
Force [kN]	17,42	34,83	52,25	69,66	87,05	103,9	118,3	126,4	131,6	136
Displacement [mm]	0,009998	0,02	0,02999	0,03999	0,04999	0,05999	0,06999	0,07999	0,08999	0,09999
Load step	11	12	13	14	15	16	17	18	19	20
Force [kN]	135,3	134,3	131,9	122,3	112,6	56,23	51,45	48,57	45,47	42,78
Displacement [mm]	0,11	0,12	0,13	0,14	0,15	0,16	0,17	0,18	0,19	0,2
Load step	21	22	23	24	25	26	27	28	29	30
Force [kN]	40,61	39,63	37,96	36,33	34,81	33,4	32,12	30,98	30,05	29,25
Displacement [mm]	0,21	0,22	0,23	0,24	0,25	0,26	0,27	0,28	0,29	0,3
Load step	31	32	33	34	35	36	37	38	39	40
Force [kN]	28,51	27,8	27,12	26,52	26,01	25,56	25,17	24,83	24,52	24,23
Displacement [mm]	0,31	0,32	0,33	0,34	0,35	0,36	0,37	0,38	0,39	0,4
Load step	41	42	43	44	45	46	47	48	49	50
Force [kN]	23,96	23,68	23,43	23,18	22,95	22,72	22,5	22,29	22,1	21,9
Displacement [mm]	0,41	0,42	0,43	0,44	0,45	0,46	0,47	0,48	0,49	0,5
Load step	51	52	53	54	55	56	57	58	59	60
Force [kN]	21,69	21,47	6,472	6,448	6,658	6,856	7,04	7,213	7,389	7,561
Displacement [mm]	0,51	0,52	0,53	0,54	0,55	0,56	0,57	0,58	0,59	0,6
Load step	61	62	63	64	65	66	67	68	69	70
Force [kN]	7,736	7,895	8,055	8,202	5,776	5,643	5,565	5,557	5,506	5,469
Displacement [mm]	0,61	0,62	0,63	0,64	0,65	0,66	0,67	0,68	0,69	0,7
Load step	71	72	73	74	75	76	77	78	79	80
Force [kN]	5,371	5,342	5,341	5,31	5,281	5,256	5,227	5,197	5,171	5,116
Displacement [mm]	0,71	0,72	0,73	0,74	0,75	0,76	0,77	0,78	0,79	0,8
Load step	81	82	83	84	85	86	87	88	89	90
Force [kN]	5,106	5,068	5,036	5,002	4,969	4,942	4,895	4,881	4,861	4,821
Displacement [mm]	0,81	0,82	0,83	0,84	0,85	0,86	0,87	0,88	0,89	0,9
Load step	91	92	93	94	95	96	97			
Force [kN]	4,798	4,766	4,736	4,708	4,687	4,665	4,651			
Displacement [mm]	0,91	0,92	0,93	0,94	0,95	0,96	0,97			

Figure 39: Force - Displacement curve based on tensile test simulation



Load step	1	2	3	4	5	6	7	8	9	10
Tensile stress [Mpa]	0,0121	0,0242	0,0363	0,0484	0,0605	0,0722	0,0822	0,0878	0,0914	0,0944
Displacement [mm]	0,009998	0,02	0,02999	0,03999	0,04999	0,05999	0,06999	0,07999	0,08999	0,09999
Load step	11	12	13	14	15	16	17	18	19	20
Tensile stress [Mpa]	0,0940	0,0933	0,0916	0,0849	0,0782	0,0390	0,0357	0,0337	0,0316	0,0297
Displacement [mm]	0,11	0,12	0,13	0,14	0,15	0,16	0,17	0,18	0,19	0,2
Load step	21	22	23	24	25	26	27	28	29	30
Tensile stress [Mpa]	0,0282	0,0275	0,0264	0,0252	0,0242	0,0232	0,0223	0,0215	0,0209	0,0203
Displacement [mm]	0,21	0,22	0,23	0,24	0,25	0,26	0,27	0,28	0,29	0,3
Load step	31	32	33	34	35	36	37	38	39	40
Tensile stress [Mpa]	0,0198	0,0193	0,0188	0,0184	0,0181	0,0178	0,0175	0,0172	0,0170	0,0168
Displacement [mm]	0,31	0,32	0,33	0,34	0,35	0,36	0,37	0,38	0,39	0,4
Load step	41	42	43	44	45	46	47	48	49	50
Tensile stress [Mpa]	0,0166	0,0164	0,0163	0,0161	0,0159	0,0158	0,0156	0,0155	0,0153	0,0152
Displacement [mm]	0,41	0,42	0,43	0,44	0,45	0,46	0,47	0,48	0,49	0,5
Load step	51	52	53	54	55	56	57	58	59	60
Tensile stress [Mpa]	0,0151	0,0149	0,0045	0,0045	0,0046	0,0048	0,0049	0,0050	0,0051	0,0053
Displacement [mm]	0,51	0,52	0,53	0,54	0,55	0,56	0,57	0,58	0,59	0,6
Load step	61	62	63	64	65	66	67	68	69	70
Tensile stress [Mpa]	0,0054	0,0055	0,0056	0,0057	0,0040	0,0039	0,0039	0,0039	0,0038	0,0038
Displacement [mm]	0,61	0,62	0,63	0,64	0,65	0,66	0,67	0,68	0,69	0,7
Load step	71	72	73	74	75	76	77	78	79	80
Tensile stress [Mpa]	0,0037	0,0037	0,0037	0,0037	0,0037	0,0037	0,0036	0,0036	0,0036	0,0036
Displacement [mm]	0,71	0,72	0,73	0,74	0,75	0,76	0,77	0,78	0,79	0,8
Load step	81	82	83	84	85	86	87	88	89	90
Tensile stress [Mpa]	0,0035	0,0035	0,0035	0,0035	0,0035	0,0034	0,0034	0,0034	0,0034	0,0033
Displacement [mm]	0,81	0,82	0,83	0,84	0,85	0,86	0,87	0,88	0,89	0,9
Load step	91	92	93	94	95	96	97			
Tensile stress [Mpa]	0,0033	0,0033	0,0033	0,0033	0,0033	0,0032	0,0032			
Displacement [mm]	0,91	0,92	0,93	0,94	0,95	0,96	0,97			

Figure 40: Stress - Displacement curve based on tensile test simulation

XI. References

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