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ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

Paula Rinaudo

Investigation of a Faulty Historical Structure: Saint Barbara Church

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DECLARATION

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

I hereby declare that the MSc Consortium responsible for the Advanced Masters in Structural Analysis of Monuments and Historical Constructions is allowed to store and make available electronically the present MSc Dissertation.

University:	Czech Technical University in Prague
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To my parents and my sister the three pillars of my life.

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ABSTRACT

This thesis faces the study of the Saint Barbara Church, which is located in Kutná Hora. In the first part of the work, a research on the historical background was carried out including the development of the city and the construction steps of the church. To understand the current state of the church it was vital to revise the historical intervention as well as the current ones.

The second part of the thesis focused on the structural behavior of the church. Basically, the aim of the study was to analyze the buttresses since the stability of Saint Barbara strongly depends on them. Due to the location of the church and its damage pattern, special attention was paid to the temperature effect in the structure, particularly in the flying arches. To study the buttresses two non linear models were prepared using finite elements. The first model consisted on the buttress in its original shape and the second one considered the strengthened system. In order to define the materials in the numerical model, compression and tensile test were carried out on the laboratory. The models were run under four different load combinations, each of them include selfweight and one temperature load case. In that way it was possible to evaluate the behavior of the structure under possible extreme temperature situations and to compare between the responses of the two models.

Supported on the results of the models and the visual survey, it was concluded that at the present the structure has a good static situation. It is remarkable that the current restoration follow the modern conservation criteria, being completely respectful of the structure. Some possible further interventions aiming to maintain the achieved state of conservation are also suggested.

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RESUME

Diplomová práce se zabývá statickými poruchami chrámu sv. Barbory v Kutné Hoře.

První část se zabývá historií chrámu a rozvojem města, který úzce souvisí s jeho stavbou. Historii a různé zásahy a opravy, které byly provedeny v minulosti, je nutné znát i pro zhodnocení současného stavu chrámu.

Druhá část se soustředí na statické chování konstrukce, zejména opěrného systému chámu, který zajišťuje celkovou stabilitu objektu. Vzhledem k charakteru poruchy a proměnnosti klimatických podmínek je hlavní cílem analýza teplotního vlivu. Byly udělány dva modely pro výpočet metodou konečných prvků s nelineárními materiálovými vlastnostmi. První model representuje původní stav a do druhého je v rámci opravy vložena vyztuž. Materiálové charakteristiky byly získány pomocí tlakových a tahových laboratorních zkoušek. V souladu se skutečností byly zvoleny čtyři kombinace zatížení, každá obsahovala vlastní tíhu a zvolené teplotní zatížení. Nakonec bylo uděláno zhodnocení obou konstrukcí.

Z výsledků výpočtů a pozorování in situ vyplývá, že konstrukce je nyní staticky zabezpečená. Důležité je také to, že oprava a restaurační práce respektovaly konstrukci a nenechaly na ní žádné viditelné stopy.

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RESUMEN

Esta tesis afronta al estudio de la Iglesia de Santa Bárbara que esta localizada en Kutná Hora. En la primera parte de este trabajo, se realizó una investigación sobre los antecedentes históricos incluyendo el desarrollo de la ciudad y las etapas constructivas de la iglesia. Para poder entender el estado actual de la iglesia fue necesario revisar tanto las intervenciones históricas como las actuales.

La segunda parte de esta tesis se focaliza en el comportamiento estructural de la iglesia. Básicamente, el objetivo del estudio fue analizar los contrafuertes ya que la estabilidad de Santa Bárbara depende fuertemente de ellos. Debido a la ubicación de la iglesia y a su patrón de daños, se debió prestar especial atención al efecto de la temperatura en la estructura, particularmente en los arbotantes. Para estudiar los contrafuertes se prepararon dos modelos no lineares usando elementos finitos. El primer modelo consiste en los contrafuertes en su forma original y el segundo considera el sistema reforzado. Con el fin de definir los materiales en el modelo numérico se realizaron pruebas de compresión y tracción en el laboratorio. Los modelos fueron ejecutados bajo cuatro combinaciones de cargas diferentes, cada una de ellas incluye peso propio y un estado de carga de temperatura. De este modo fue posible evaluar el comportamiento de la estructura bajo situaciones de temperaturas extremas y a su vez comparar las respuestas de los modelos.

Basándose en los resultados de los modelos y la inspección visual se pudo concluir que la estructura presenta actualmente una buena situación estática. Es destacable que la restauración actual sigue los criterios de conservación vigentes siendo totalmente respetuosa de la estructura. Se sugieren algunas intervenciones complementarias con el objetivo de mantener el estado de conservación alcanzado.

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TABLE OF CONTENTS

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1. INTRODUCTION

The cathedral of Santa Barbara [Figure 1] is located in the city of Kutná Hora 70km from Prague. Its construction started in 1388 with the purpose of finishing the fight for the own religious autonomy between Kutná Hora mine owners and the Sedlec monastery (under the Cistercian order).

Figure 1 Saint Barbara Cathedral

The cathedral was dedicated to the Virgin Saint Barbara, the patron of the miners. It was set up as a manifestation of the artistic and cultural aspirations of Kutná Hora patricians, which pretend to build an edifice that, surpass all the churches in Kutná Hora and equal Prague cathedral. It also expresses the importance and power of the town, developed around the seventies of the 13th century by the mining communities.

The construction of the cathedral progressed in several stages and depended greatly on the prosperity on the local mines. The work was interrupted numerous times under the influence of various events, and more than 500 years passed between commencement and completion of the work in 1905. As a result it is a product of gradual and mixed dynamic evolution, attributing highly remarkable vaulting systems and an interior furnished with medieval frescos pointing out the live of the miners. The church is consider as “a jewel of the late Gothic period and had a profound influence on subsequent developments in architecture of central Europe” [25].

For all those reason the Cathedral of Saint Barbara was included in UNESCO world cultural and natural heritage list in 1995, as well as the historical city center of the town and some other buildings.

Nowadays Saint Barbara cathedral has mess only once a week. However the church is one of the city principal attractions from the touristic point of view. Because of that, it is essential for the economical activity of the city to preserve this building.

Due to the wheatear of Czech Republic, the structure has to resist cycles of temperature loadings. This structure is made of masonry and tends to adapt to the temperature variation through deformation and the development of cracks in the flying arches of the buttresses. However, the impact of temperature variation on the structure has not been under study until the last years.

The church has undergone historical and actual restorations. It is important to highlight that the flying arches were repaired many times since they have been built. Moreover at the end of the 19^{th} century some calculation based on graphic methods were done. As a result, some modifications in the shape of the upper arch and pier of the buttresses were done at that time. Those changes were not much successful since they did not consider the real cause of damage. The flying arches were lately reinforced as part as the current restoration. The scope of this work is to evaluate the current state of Saint Barbara church and in particular the condition of the flying arches.

2. DESCRIPTION OF THE CITY [19]

The town of Kutná Hora is located about 70km East Prague at 254 m above the level of the Baltic sea at the steep descent of the Vrchlice valley. This valley is the natural limit of the historical center of the city in both south and south-eastern sides. The east part of the city is called the Lower Part and ends where the valley rise up to the north, where is the Upper Town.

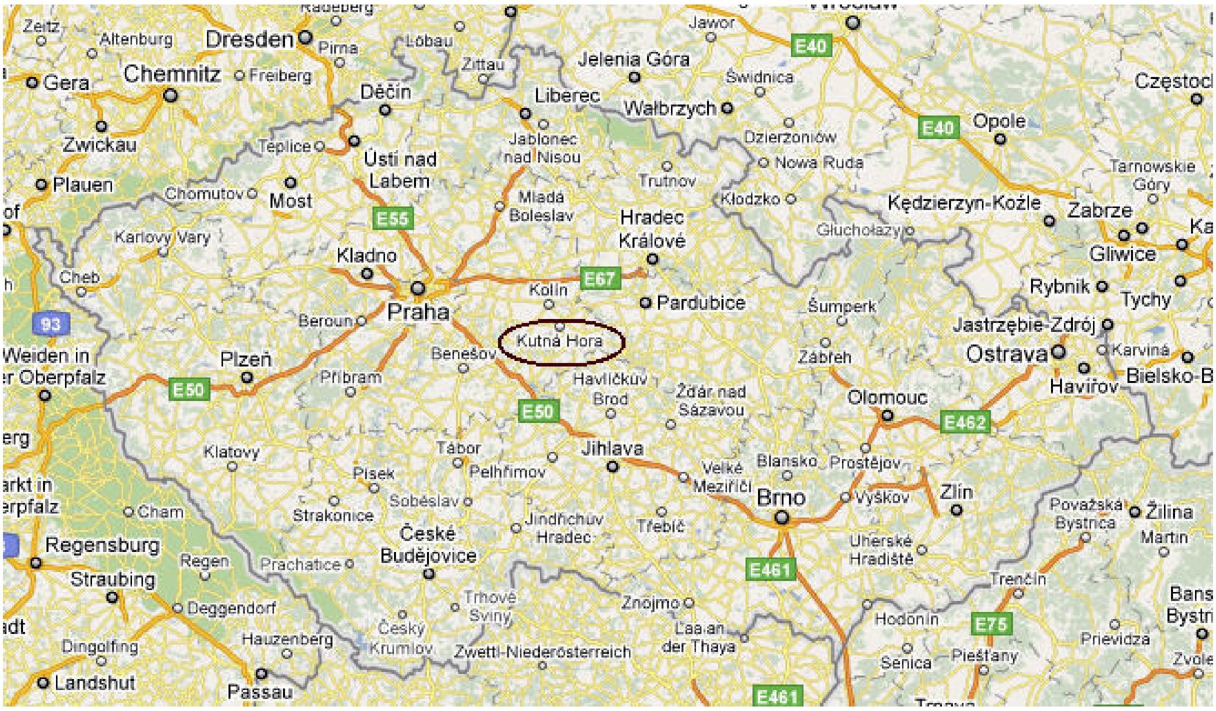

Figure 2 Location (Source Google maps)

A map of the historic center is shown in Figure 3. Important buildings are marked by big numerals. In the legend above the figure their names are listed, as well as the names of the disappeared buildings and gates. The shape of the existing city walls are highlighted in the map. In the figure it can be observed that the Cathedral of Saint Barbara was located outside the city walls.

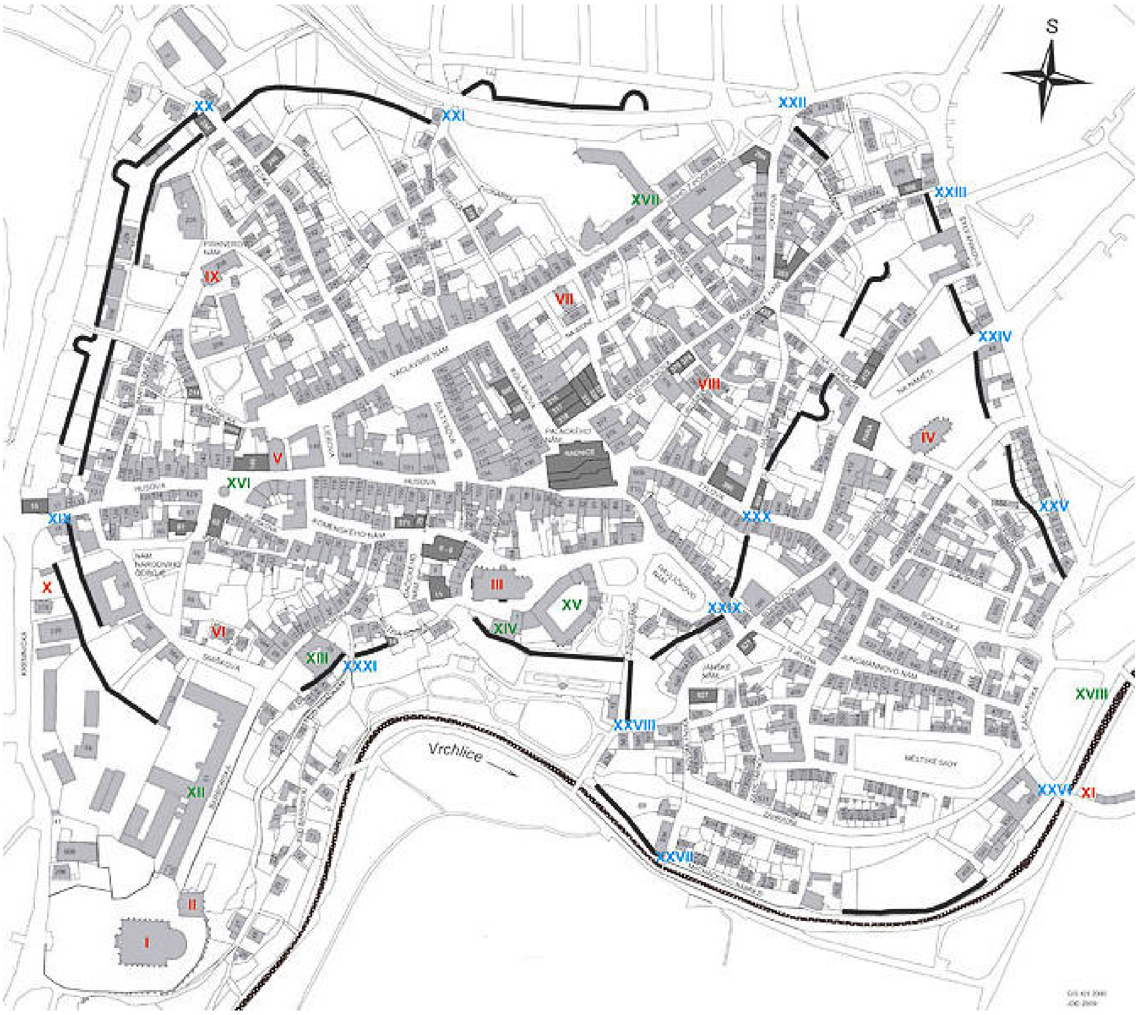

Figure 3 Map of the historic center [19]

Churches and chapels

I St. Barbara Cathedral

II Chapel of Corpus Christi

III St. James Church

IV Church of Our Lady Na Náměti

V St. John of Nepomuk church

VI Hieron of the Czechoslovak Hussite Church

VII Hieron of the Czech evangelical church

VIII Former chapel of the Holy Trinity

IX Former St. Bartholomeus church

X Disappeared St.George church

XI Disappeared church of the Holy Cross

Monumental buildings

XII Jesuit College

XIII Small Castle

XIV Archdeanery

XV Italian Court (mint and royal residence)

XVI Stone Fountain

XVII Ursuline monastery

XVIII Executioner´s house

Disappeared gates

XIX Kouřim´s gate

XX Kolín gate

XXI Hlouška gate

XXII Monastery gate

XXIII Sedlec gate

XXIV Nová (New) gate

XXV Executioner´s gate

XXVI Čáslav gate

XXVII Novomlýnská (New mill) gate

XXVII Žižka´s gate

XXVIX Leflířská (Cobblers´) gate

XXX Stará (Old) gate

XXXI Podhrádecká gate

3. HISTORICAL BACKGROUND

3.1 History of Kutná Hora [16][17][19][25]

The town was developed as a result of the exploitation of the silver mines in the 13th Century. Around the 1260 the silver ore was discovered by prospectors that systematically surveyed the area of Českomoravská Vrchovina highlands. It was the latter that determined the earliest occupation in what is now the historic centre of the town, which have been occupied by numerous scattered mining settlements in the 13th centuries. This early exploitation of the mineral resources explains the resulting complex street plan of the city.

The city plan conserves at its core a non urban road conjunction compose by a road leading from Prague to Malin and other to áslav. There is although a third road, which leads to Kolin. The location of these ancient settlements over the city can be seen in Figure 4.

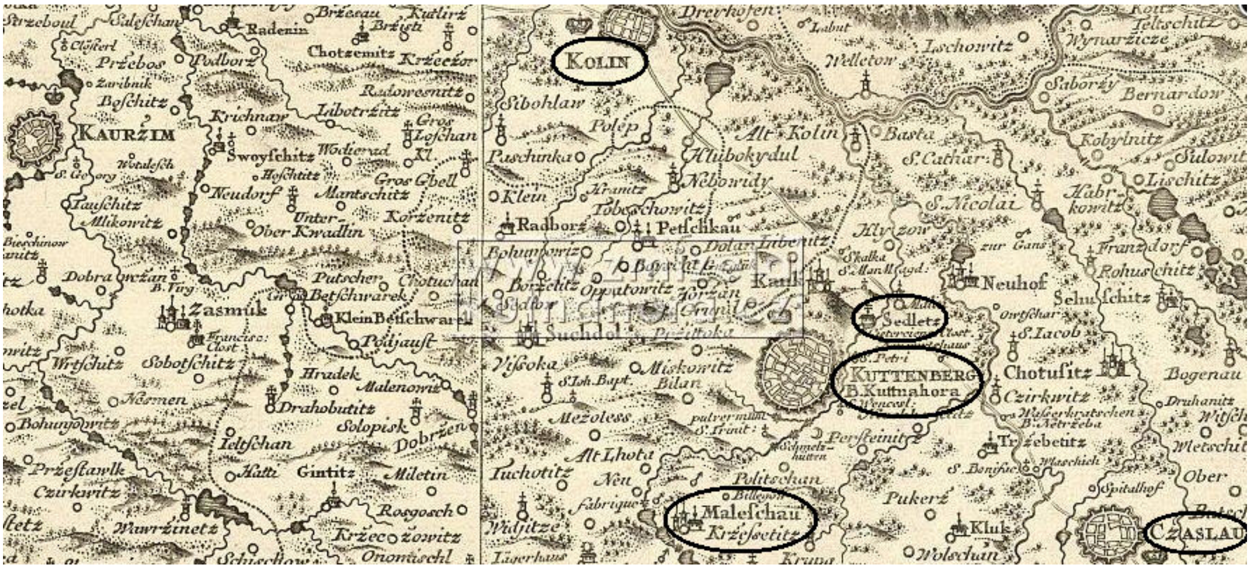

Figure 4 Historical map of Kutná Hora (1720)[19]

The oldest archeological remains of processing of silver were found in Malin, there were produced coins in the 10 century. The importance of Malin sunk after the wipe out the nobility family Slavníkovci in the year 995.

The Cistercian monastery in Sedlec was established in the year 1142 by the king Wenceslas II (Vladislav II) and aristocrat Miroslav and adopted the position of principal spiritual authority and supervised the formation of legal and administration system.

The royal fortified towns of Časlav and Kolín were founded in the early 1260s, both closely associated with the silver mining in the area. All legal affairs had to be dealt with the latter mentioned neighboring towns.

Firstly, the mining settlements were called just Mons (mountain or hill) and the more specific name of MONS CUTHNA appeared only in 1289. New settlers, mostly from neighboring German speaking regions, were attracted by rumors about rich silver deposits. These settlers become the leading group in the entire agglomeration, because of the advanced manufacturing technology and social system that they implement. At the beginning, the mining settlements appear without any plans and system and no rights were guaranteed.

In 1300 the new royal mining code IUS REGALE MONTANORUM was issued by the King Wenceslas II (1285-1305). This vital document specified not only all administrative terms more also the technical terms and conditions necessary to explode the mines. Wenceslas II, also made the reform of the coinage and replaced the various coins produced in diverse cities in a new single currency, the "Prague groschen", which were produced in the mint established at the Vlašský dvur (the Italian Court). This currency was one of the most stable in Europe. Kutná Hora thus became the country's most important economic centre. The chaotic cluster of miner's huts was transformed progressively into the second most important town of the Czech Kingdom, after Prague. At the same time it was being transformed into a royal town, with all the rights and privileges to be confirmed later by King Jan Lucemburský and King Charles IV.

To protect the city against invasions of the Roman Emperor Albert I, who wanted to empower Kutná Hora, provisional wooden town walls were built around 1304. After that, they were gradually replaced by real city walls of massive stone, which had to be extended after some time, due to the rapidly expansion of the metropolitan area. By the middle of the 14 century the definitive defenses walls were complete. Considering medieval standards, the walled-up area was huge indeed, comparable to the Old City of Prague.

From 1320s, the rise of the city was accompanied by the construction of monuments that symbolized its prosperity. The original wooden sanctuaries were overcome by more important structures. The High Church began to be built (now a days called St. James) and later the Church of Our Lady Na Náměti and others that have not survived. The city gradually went on constructing social facilities.

The dispute between Kutná Hora and the Cistercian monastery in Sedlec became stronger during the time until the end of the 14th Century when the self-confident municipal authorities led to the decision to break away from the monastery and its administration. The decision to build St. Barbara away from the jurisdiction of the monastery, made around 1380, was the manifestation of emancipation based on seemingly inexhaustible deposits of silver.

At the same time, the mining activities were facing the first serious problems since silver deposits near surface were finished and more sophisticated techniques were necessary to descend deeper under the surface. As a consequence, both mining and production of coins were inhibited for some time.

The Hussite wars saw profound changes at Kutná Hora. During that time the city was dominated by German patriciate, which governed the town's attitude along the Hussite Wars. At the beginning, the city stood firmly behind Emperor Sigismund. However after a series of Hussite triumphs in the course of 1421, Kutná Hora lost all hopes for keeping the warfare outside its limits. The monastery in Sedlec was conquered and burnt down in May 1421. There were two devastating fires in the city in 1422 and 1424 which destroy most of the buildings. Bu this time, the people which wanted to remain catholic had to leave the city. The consequences of those events led to the collapse of mining activities until the last years of the reign of George of Poděbrady. In 1469, an increase in silver production was generated. Mining was renewed with enormous difficulties and Kutná Hora again became the scene of political events of national importance and an economic mainstay of the Bohemian Kingdom.

In 1471, an assembly meeting took place in the Italian Court, resulting in the election of Polish king Vladislav II Jagello as king of Bohemia. Kutná Hora was in an outstanding period where the mining activities were fully restored. The coin production was large enough to invest in the construction of Prague Castle and in buildings in Kutná Hora.

In the early decades of the 16th century, Kutná Hora seemed to have inexhaustible wealth with mining zones producing reasonable yields. Though, miners had to descend about 500 meters under the surface and the risk of being flooded by ground water was increasing. The other alarming situation was that the thickness of silver ore was decreasing. This situation became worst in 1530s and the mining activity was restricted in the main bands. In 1543 the most famous mining zone, Osel, closed since there were not sources to invest in new technology. By 1547, the production of Prague Groschen in the Italian Court arrived to the end. During the 16th century the city had a favorable situation. The mining activity continued at Kaňk zone, this mint was working at full capacity thanks to imported silver and the town showed sings of overall prosperity.

At the early 17th century, the crisis increase and it was clear that there was no solution to maintain the wealth on Kutná Hora. The defeat of the Battle of the White Mountain in the year 1620 occurred at the same time; consequently the religious freedom was reduced. During the first months after the battle, the progress of re-catholicization was still moderate and municipal authorities try to preserve freedom of religion for German Lutheran settlement near St. George in order to prevent mass emigration of miners and consequent lack of labor forces in the remaining mining zones.

Imperial authorities were not satisfied with the slow progress of re-catholicization of Kutná Hora. As a consequence, in 1625 they committed the infamous troops of Colonel Huerta to speed up the process. Moreover, the Jesuits arrived in Kutná Hora and with the support of Mr. Vřesovec, Master of the Mint, decided to build their college there.

Along the 30 years of war Kutná Hora over passed unimaginable problems. A permanent increase in taxes and contributions confiscation of suburban states and troop movements caused the ruin of the municipal Budget.

Mining was not able to overcome the situation and was terminated. The deficient finance of the city obliged the town council to request permission to leave the Kaňk zone. At that time the mint was producing using mainly imported silver. After much effort, 1628 the town recover its privileges, however, only to the Catholic citizen enjoy the freedom.

In 1639 and 1643 two Swedish invasions affected the city, causing extensive damage. At that time the city was full of empty houses and also the mines were abandoned remaining only a few miners.

At the end of 1650s first signs of economic recovery were observed. The development of crafts and trade was encouraged by peaceful relations. Also first attempts to bring mining back to life appeared. The recovered financial situation enabled gradual re-acquisition and extension of municipal estates outside the city walls. Spiritual life was in the hands of the Jesuits. Their School should be considered a positive achievement, because it was providing a fair amount of knowledge and educated many individuals that found their fulfillment in the order as well as outside of it. In the painting by Jiří Čáslavský [Figure 4] we can observe the oldest shape of the town and individual buildings.

Figure 5 Vista of Kutná Hora by J. Caslavsky (1674) [19]

The intention of opening new mines and restoring the old fame of Kutná Hora as a mining town was installed during the 18th century. However it was not possible since new silver veins were thin and their extraction required unaffordable cost. Such situation ended up in the closing of the mint in 1727. It can be said that the era of the mining town was over, although mining authorities still kept their seat in the Italian Court.

The city of Kutná Hora was still ranked among bigger towns in Bohemian context at the beginning of the 19th century with approximately six thousand inhabitants in 1800. Nevertheless its significance was inexorably lost. For a long time, there were only two industrial plants. One established in 1774, were the Breuer Cotton-print Works and the other was the National Tobacco Factory founded in1812 in the Cistercian monastery.

The main components of Kutná Hora of that time consisted mostly of clerical intelligentsia, military officers and middle-class craftsmen and businessmen. They were also responsible of the construction activities, particularly after 1823, when a huge fire destroyed most houses in the eastern part of town.

In 1877 the Vocel Archeologist Committee was founded and it allowed the conservation and renovation of many historical buildings in Kutná Hora. At that time some considerations regarding the conservation of historical values were discus. The main discussion was, preserving the town as a whole or selecting only the most outstanding monuments. Due to the lack of economic sources, those debates were only at academic level. Neo-gothic purism was in opposition to “jesuits Barroque”, and as a consequence some Barroque buildings were demolished. The sources available were invested mainly in the most damaged building including St. Barbara’s Cathedral, which was in critical state. Another important building in a deeply damage state was the Italian Court.

With the new republic of Czechoslovakia some citizens of Kutná Hora believe that they would succeed in enlarging the town by new, modern quarters, and as a result established conditions for more dynamic development. Municipal authorities try hard to encourage industry, crafts and trade and to attract younger population. Unfortunately they were stuck due to the unfavorable development of transport, especially of the railway system.

Kutná Hora therefore remained a town of monuments and tourists. Fortunately, the more recent historical turbulences have had no significant impact on historic and cultural features of the town.

In December 1995, the historic center, as well as Saint Barbara Cathedral and the Cathedral of the assumption in Sedlec were included in UNESCO World Heritage List.

3.2 Construction history [1] [3] [21] [27]

The cathedral symbolizes a unique example of an urban church, on which was focused lot of attention of the patriciate of the town. Saint Barbara cathedral was built in several phases. It can be notice that it does not follow the monastic regulations, as is illustrate in its decorative elements and their colors.

Construction work began in 1388 above Vrchlice river valley. The church dedicated to the patron of the miner, Saint Barbara, is situated outside the city wall. The reason for that was mainly the intention to separate the spiritual administration of the town from the Sedlec monastery. In addition, the density of the town did not allow finding enough space inside the town wall to build the church. Moreover, the place was also chosen because there was a small chapel dedicated to Saint Barbara.

The first designer of the Cathedral was probably John Parler [1](son of Peter Parler who built St. Vitus Cathedral in Prague). Initially the design was a triple nave type and the church it self was of a greater length. Nevertheless, it was soon changed into a five nave building.

The oldest part of the cathedral is the presbytery. Until 1420 its ground storey was raised below the cornice of the triforium. During and after the period of the Hussite war the construction of the building was interrupted more than 60 years. The work was continued by Master Hans, 1482 to 1489, following Parler design. By this time the work on the triforium and on the outer buttresses was begun.

In 1494, Matyas Rejsek[2] arrives in Kutná Hora to go on with the works of the church. He completed the external buttresses and in 1499 vaulted the outer aisles and the gallery round the presbytery. He worked in Kutná Hora until his death in 1506.

Figure 6 Vault by Matyas Rejsek

The next construction phase of the cathedral was done under Benedikt Ried[3] directions, which designed the second floor of the nave and its monumental vaulting with helical ribs.

[1] Johann Parler (1359? –1406)Johann Parler continued Peter Parlers' work on St Vitus Cathedral. He later moved to Kutna Hora, and was likely the first architect of St Barbara Church, of which Peter Parler participated in the draft design.

[2] Matěj Rejsek (1445? – 1506). He was a Czech architect who worked on the south tower of the Church of Our Lady of Tyn, which was finished in 1511, and the architect of Powder Tower in Prague.

[3] Benedikt Ried (1453 – 1534) He was a German architect, who worked in Prague Castle and in St. Barbara Church in Kutna Hora (1482).

Above the pillars of the arcades he erected slender concave pillars, from which stands the ribs of the vault with six and four – pointed stars, leading to vaults that do not follow the traditional gothic style.

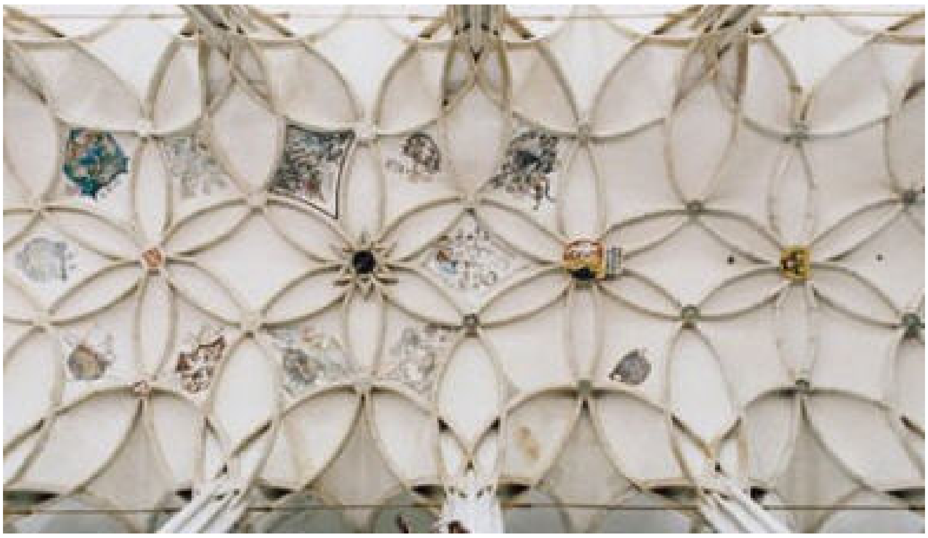

Figure 7 Vault by Benedikt Ried

The roof with a three tent shape was built after Benedikt Ried death. In 1595 the cathedral was completed with the construction of a temporal wall [Figure 8]. By that time the silver mines were gradually exhausted, which means lack of maintenance of the cathedral. In fact the temporary wall remains there until 1893.

Figure 8 Temporary wall a) 1884 b) 1893 [19]

In 1626 the cathedral was under Jesuits administration. They adapted the church for its college chapel in a baroque style and in 1667 began to build the College itself, which was connecting the cathedral with the town symbolically and practically. The more significant once were the replacement of the tent roof with a hipped roof (1734) [Figure 9] and the building of a suspended roofed gallery to connect the cathedral with the college.

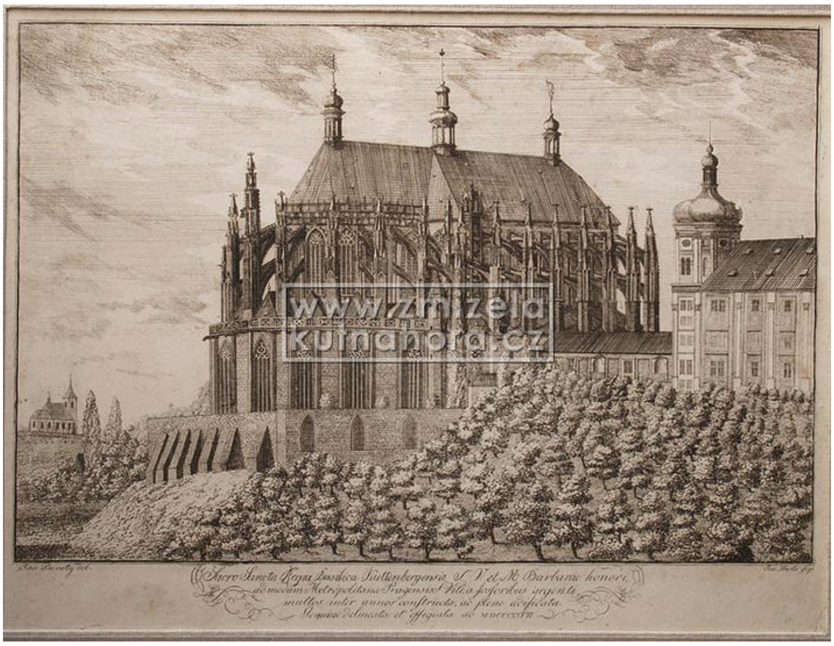

Figure 9 Saint Barbara cathedral with the hipped roof [19]

The interior of the church was also modified since the original interior equipment was removed. Upon abolition of the Jesuit order in 1773, the cathedral fell to the state religious fund and began to fall into decay again since there was no maintenance and repairs.

Thanks the foundation of the archeological society Vocel in 1877, the cathedral was not only saved of further deterioration more also restored to its original form. The latter restoration started in 1889 and was in a Neo-gothic style. Restoration and reconstruction project was created by the architect Josef Mocker[4]. During this works, the baroque modifications were eliminated, the gallery connected to Jesuits College was demolished and the tent roof was set back (1897-1899) [Figure 10].

[4] Josef Mocker (1835 - 1899) He was a Bohemian architect and restorer who worked in a purist gothic style. He was responsible for restoring many Bohemian castles, churches and ancient buildings in Prague. His work aroused much controversy.

Figure 10 Restoration of the tent roof 1897 [19]

Also almost all the furnishings of the building were eradicated. Gothic elements were renewed and newly supplemented rather than restored. The exterior chorus has been renovated between 1884 and1889. The temporal wall finished in 1595 was changed in 1893 by an entire new façade. The western frontage was built as well as one more vault to enlarge the west part for static reasons. Particularly attention was paid to stressing the Gothic elements, specifically to the buttresses. The ones of the nave that were more damaged had been substituted using new stones and a bit changed shape. Also, the vaults and presbytery buttresses were renovated. The cathedral was finally consecrated in 1905.

4. GEOMETRICAL DESCRIPTION [3] [9] [17]

The temple is 70 m long by 45 meters wide and has a variable height. The façade is 40m tall and the tent roof is around 70 meters high. The church has a plan design compose by three naves [Figure 11]. The total span of the central nave is 23 meter, divided into 6,5 m in each aisle and 10 meters in the central part, while the span of the laterals naves 8 meters. The vaults of the main nave ascend up to 5 meters above the height of the central nave, while the vaults of the laterals naves climb 3 meters from the height of side naves. The thickness of the vaults is 0, 3 meters.

The interior of the central nave has 22 columns, which diameter is 1,7 meters and its high is 24 meters. Those columns define on both side five aisles and on its upper level tribunes.

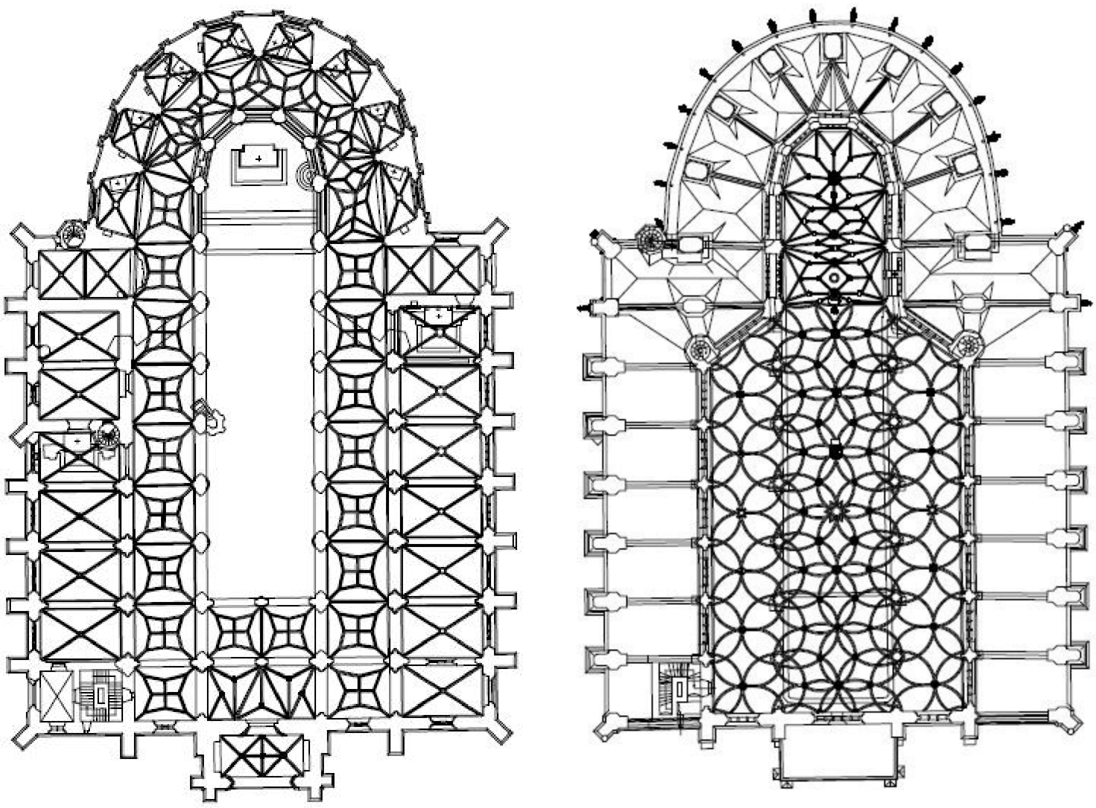

Figure 11 a) Plan b) Vaults

The transept is represented at bought laterals by porches. The sacristy is after them on the north side; on its top floor the treasury is placed.

The cathedral choir is located at the east end of the main nave [Figure 13 a]. Its external perimeter is almost round, including 15 sides of a 28 side polygon, while the interior contain five sides of a decagon. In the core of the choir is located the presbytery rounded by ambulatory and radial chapels.[3] The latter have trapezoidal shape and gothic vaults. Between the chapels there are deep pillars to rectify the geometrical disparity among the inside and outside perimeter of the choir.

Buttresses were placed deep between the chapels. They can be describing as a massive pier and two flying arches with different sections, where the upper one has a higher modulus of inertia than the lower one.

Figure 12 shows the cross sections for the lower and upper arch of the nave. For each section the value of its area is written. It can easily notice that the areas of the sections that correspond to the lower arches have approximately the half of the area that match to the upper arches.

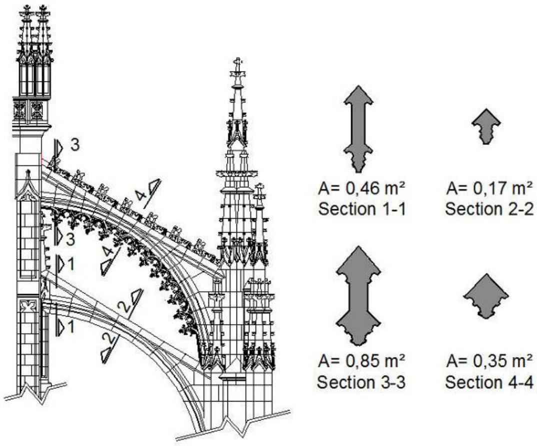

Figure 12 Sections of the flying arches

The perimeter wall of the building is distinguished by wide windows with gothic shapes. In the upper part of both, the perimeter wall and the pillar are placed pinnacles [Figure 13 b].

Figure 13 a) Interior east view b) Exterior south view

From the architectural point of view, the cathedral is an example of Gothic architecture, with a unique combination of ribbed vaults, pointed arches and flying buttresses. Those elements help to emphasizes verticality and light. Gothic openings such as doorways, windows, arcades and galleries have pointed arches.

5. INCLUSION IN WORLD HERITAGE LIST [23][24] [25]

In 1995 the Church of Saint Barbara was included in the Word Heritage List as part of a group of buildings, which were defined under the title: "Kutná Hora: Historical Town Centre with the Church of St Barbara and the Cathedral of Our Lady at Sedlec" (World Heritage Center 732) [25].

UNESCO makes reference to this church as "a jewel of the late gothic period" and as "a masterpiece that nowadays forms part of a well preserved medieval urban fabric with some particularly fine private dwellings". In UNESCO documents it is also highlighted that the cathedral influenced the architecture of central Europe.

Kutná Hora was evaluated as a city that has a considerable measure of uniqueness. Its urban landscape is considered one with a high aesthetic quality.

Saint Barbara is recognizing as a building of exceptional interest in terms of both ground plan concept and architectural design which results from "gradual, highly varied evolution, featuring highly remarkable vaulting systems and interior furnished with an immense wealth of art objects".[25]

ICOMOS underlined that this cathedral has particular architectural and artistic quality.

Regarding the protection of the building, it was also declared National Cultural Monument and a program of regeneration of the town was also developed. The city of Kutná Hora celebrates this year 50 years since it was declared historical town reserve.

The documents of UNESCO also include the uses that are allowed in this site, like religious use and visitor attraction. Now a day, the latter is the main one.

The document written in 1995 included as well suggested conservation intervention that should be carried out, as the repair of the roof, the supporting system and the restoration of sculptural decoration. The majority of this works have already been performed in the last years. A description of the intervention carried out after the declaration of the building a World Cultural Heritage is available in the following chapter. UNESCO also prepares annual reports on the condition of the monument.

6. INTERVENTIONS AND RECONSTRUCTIONS

The interventions in this building can be undoubtedly divided in the historical ones and the currents restorations.

Regarding the formers, it can be said that the building underwent two big renovations. The first one was performed from 1626 until 1773 under the direction of the Jesuits and the second one at the end of the 19th century under the direction of Joseph Mocker and Ludvík Lálber. The main works made in both of them has already been described in the chapter about the construction history of the building.

The works did by the Jesuits show lack of restoration theories and awareness of the ancient construction as cultural value. [20] It can be define as a conversion according to the Jesuits necessities and an update to the baroque style. Dramatic transformations are the main characteristic of this intervention, among them the substitution of the roof is the clearer one. However, the stone mason Peter Baumgartner from Prague was invited by the Jesuits to make the renovation of the buttress, because of his understanding of the gothic style.

The restoration performed on the late 19th century was characterized by the idea of presenting the buildings in their original purity. [27] It can be describe as a stylistic restoration, were restoration was conceptualized as an operation of stylistic re-integration through which the architect should try to attain the unity that the monument had in its primitive condition. [20] Under this concept, liberations and demolitions of later additions, such as the Baroque ones, were legitimate. [Figure 14]

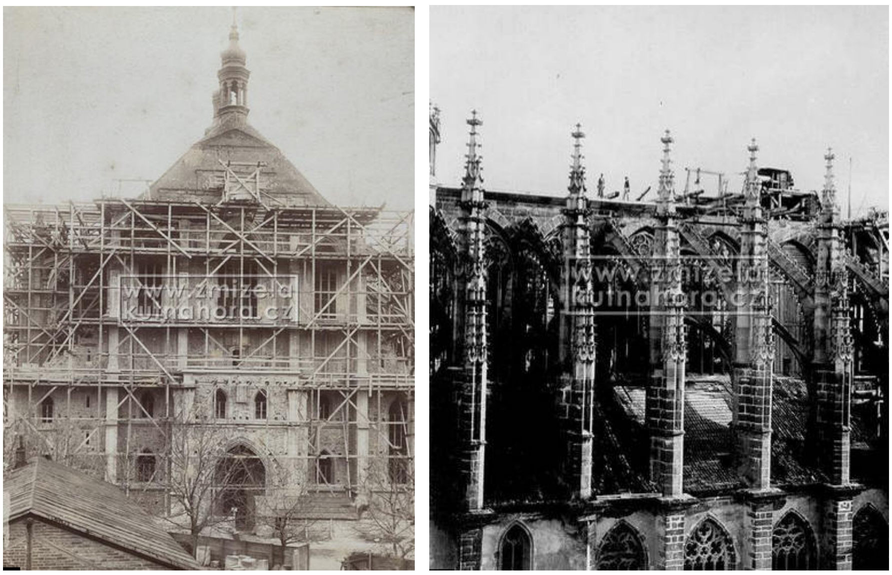

Figure 14 Change of the roof to its original shape [19]

During this time, there was public interest of the monument recognition and the need of intervention to safeguard it. The interest in the survival of a monument did not lay in the identity of its materials but in the identity of its shapes and proportions.

Due to the lack of knowledge about materials compatibility, a vital error was made during this restoration. The majority of the original limestone block of the buttresses were changed and replaced by copies made of sandstone. As well the tracery of the windows of the nave and some stones of external sculptures were replaced with the same material. The mixture of sandstone and limestone is an inadequate combination because of the different nature of the mineralogical composition and porosity of the rocks. Limestone is basically composed by calcium carbonate, that itself has not a disintegrating effect once it has crystallized. However, under the influence of meteorological conditions the calcium carbonate can be transformed to calcium hydroxide (gypsum) and can be mobilized. In this case, the water is the transport medium and solvent for the gypsum. Due to the high porosity of the sandstone, the gypsum as a solution can penetrate in it. In the process of crystallization of gypsum, big forces are generated and consequently the sandstone is damaged. Also crusts were formatted on the sandstone surfaces.

Other mistake of this restoration was that lead was placed into the joints between stones. Lead and sandstone have different thermal expansion coefficients as a result of thermal cycles; the expansion and shrinkage of the material generate cracks on the interface between lead and sandstone. In the current restoration those crack were fill with mortar, to avoid their propagation and opening, which will lead to more complicated repairs. Moreover, lead was also used in the joint of the buttresses but it did not introduce any positive effect since it has no capacity to take shear.

After the inscription of the Church as World Heritage, studies on the structure and restorations were encouraged; those restorations can be identified as the current restorations. It can be notice that they are based in the modern concept of restoration.

In 1996 the first steps of the investigations were carried out, including research on the building history and a preliminary structural analysis for a draft design for the repair. Before starting with the restoration works, photogrammetric documentation of all the different parts and details was done. In 2000 according to the results of the first researching stage, the roof was restored since its damaged condition allowed rain water to penetrate in the stone and this was deteriorating the vaults and some sculptures that have an important artistic value. The roof was covered with slate and restored too; flashings were done from cooper sheets.

During 2002 scaffoldings to perform supplementary analysis of the first section of the façade were placed. This section is called “Section 1” in Figure 16. Under the preservation aims, a temporary securing system was implemented on the buttress of this section including wedging of the opened joints and supporting of the sagged flying buttresses (Figure 17). By this stage further analysis were prepared, such as a petrologic survey and a detailed study about the construction process, construction techniques and previous restorations of stones and windows. Also, models of the faulting buttresses were carried out to clarify the causes of damaged. Base on those studies the restoration project of the buttresses was defined and during 2003 it was developed.

Figure 15 Restoration of the roof [22]

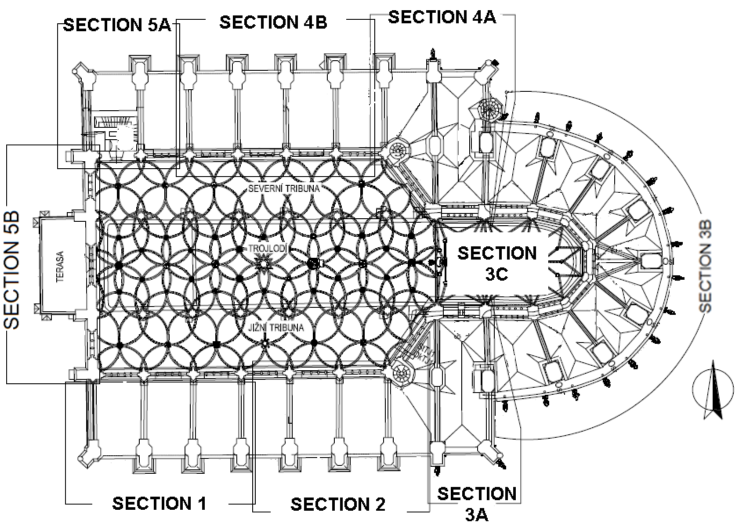

Figure 16 Restoration sections [15]

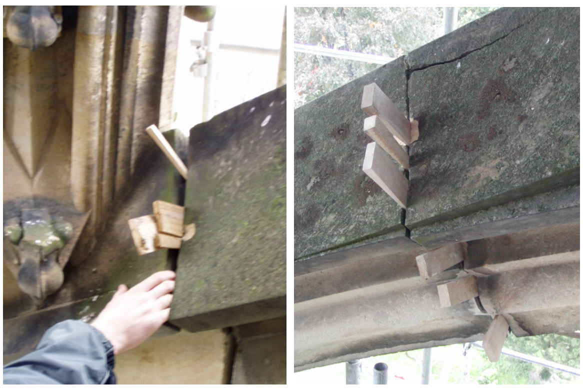

Figure 17 Temporary security [11]

The restoration implemented during 2003 on the four buttresses of Section 1 consists in connecting adjacent stones of the arches using stainless rods [Figure 18]. Every two consecutive stones, a common joint was defined trough drilling holes on the stones and then introducing the rod. Afterward, the joints were grouted with new elastic mortar. In that way an invisible repair was done. The reinforce transfer the shear forces and avoid stones to fail. Special attention was paid to the location of the roads, since they need to be near the neutral axis in order to ensure not to change the location of the forces inside the flying arches. After the restoration fifteen monitoring points were set on the buttress to observe its behavior. A complete description of this part of the restoration is available in [11].

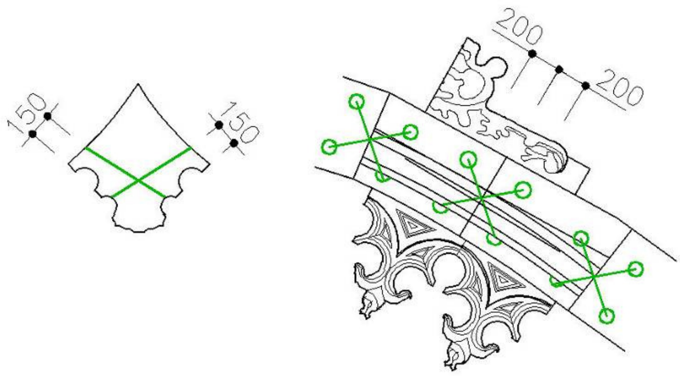

Figure 18 Reinforcement a) Section b) View [11]

The works performed on 2003 included as well the restoration of 12 mascarons that form part of the external decoration and are placed under the cornice in the corners of the chapel walls and buttresses. Those sculptures from limestone, originally made between 1521 and 1524, were damaged as a consequence of air pollution and water penetration in the cornice. The selection of the more suitable materials to restore them was based on test on different possible combinations. In the restoration process, the mascarons material was solidified and then additions were placed where necessary to guaranty the flow of water. Finally the sculptures were covered with impermeable surface material and the cornice was covered by lead sheets.

Figure 19 Mascarons a) Before restoration b) After restoration [7]

While the restoration of Section 1 was implemented, scaffoldings were placed on Section 2 to permit the study of this section. The design of this intervention also included the same method of strengthening of the buttresses used in Section 1. This section was restored in 2004.

Through the first half of 2004 scaffolding were place in section 3A to restore the transept and during the second half of 2004 scaffolding were placed in section 3B to repair the façade of the choir and the glazed windows. [Figure 20] In this section the buttresses were not extremely damaged, however since the decay of the buttresses has a cyclic pattern it was consider worth to perform a preventive restoration of them and also make the strengthening of the buttresses using the same procedure that in Section 1.

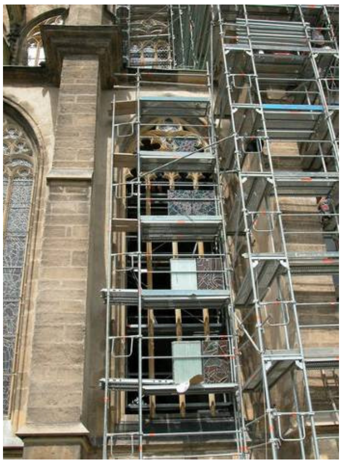

Figure 20 Restoration of the stained widows [13]

In winter 2005 scaffolding were place on section 3C to restore the interior of the choir and the glazed windows from the inner part. Restoration of the glazed windows included stone strengthening, cementing and where necessary stones additions. Trough the scaffolding on Section 3C it was possible to access the triforium and the vault of the choir in order to evaluate its current situation. This was particularly important, because in 1884 walls were build in some passages of the triforium and consequently some parts of the triforium were inaccessible. Also, the stone, plasters and painting of the triforium were analyze before the design of the intervention of the choir. During this stage it was discovered that condensation problems on the vault and on the windows mark were related to the high humidity of the choir. This was a result of the restoration of the 19^{th} century since the vents placed in the vaults were closed by this time reducing the ventilation. In the current restoration this vents were opened again. The interior surface of the vaults of this area was restored too; the plasters of the 19^{th} century were replaced by a new one with the characteristic according to durability and compatibility patterns.

To complement the studies on the church a thermal analysis was done during the years 2005 and 2006. More information about it is mention in Chapter 8 (Temperature as a cause of damage).

The restoration of the exterior part of the church continued between 2005 and 2009. Following the direction of the works, scaffoldings were placed in Section 4A and then in sections 4B and 5A covering the rest of the North surface. The buttresses of this side were also reinforced. In particular the last buttress of nave on the north side (which was part of Section 4A) was not restored in order to be able to compare the behavior of the reinforced buttresses with the one that was not reinforced.

It is important to notice that the buttress that have not been reinforce are the ones in correspondence with the transept (section 3A and 4A) and one in section 3B. The exterior stones were cleaned with water steam from the scaffolding in all the different sections.

It is vital to remark, that the current restorations show respect for cultural and heritage diversity. That could clearly be seen, for example noticing that before each stage of restoration a detailed project was done including the particular difficulties of each step. For instance for section 1 a particularly scaffolding was design to apply first the temporary security system and then the reinforcement.

The structure of the church was considered as part of the cultural heritage and consequently historical structures were conserved following authenticity criteria. Not only their appearance and materials were preserved; also their structural and strength reality.

Those restorations were accompanied by information sources that can be find the church, allowing tourist to understand the nature, specifications, meaning and history of the cultural Heritage. The current restorations works were also illustrated in presentation panels, facilitating the appreciation of cultural heritage and encouraging public to be aware of the need for their protection and conservation.

7. VISUAL SURVEY - CURENT SITUATION

A visual Survey was carried out the 29th of April 2011 guided by one of the responsible of the current restoration of the Church.

One of the outstanding values of Saint Barbara church is its external decoration allusive to miners. At the present they show a well conserved state since this decoration was considered as part of the architecture group of the church and consequently it was subjected to restoration in the last years.

The external walls are made of stones of diverse sizes. It can be notice that the naves were built with smaller stones than the sacristy. Additionally, no settlements of the walls are seen. The current state of the walls is good, showing repointing form the last restorations.[Figure 21]

Figure 21 Repointing on the exterior walls

The façade was made of stone of diverse sizes since it was erected very fast. In particular, shaped stones were used for the window jambs, spandrel, cornices and border of piles; while the rest of the masonry of the façade was done of quarry stone. The masonry made of shaped stones has thinner joints than the masonry made of quarry stone. Moreover, it has a lower quantity of joints per unit of high. As a result, the load tends to concentrate in the window jambs. It this areas the presence of small settlements can be noticed, on the contrary the spandrel of the windows do not show existence of settlements. These differences in settlements cause cracks in the spandrel of the windows and lead the load to move to the tracery. Consequently, cracks are also developed in the tracery. In Figure 22 the random size of the stones and a border from one of the windows can be seen.

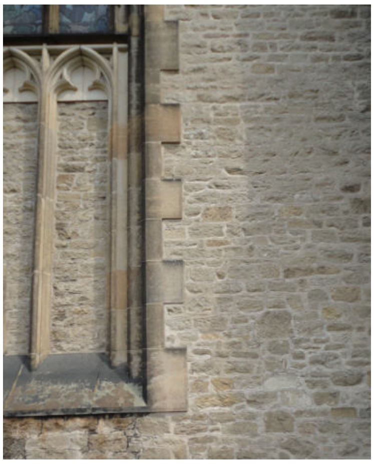

Figure 22 Window of the façade

The floor of the balcony of the façade is in a very poor condition showing a cracked pattern as a consequence of the temperature cycles. It can be deduced that during past restorations successive layers of asphalt paint were applied. The total thickness of layer can be estimated around 8 cm.

Figure 23 Asphalt layers of the balcony floor

From the outside of the building it can be notice that one lateral column on the North side has a different section [Figure 24]. This column is a clear evidence of the change in the design of the building from a triple nave type into a five nave building.

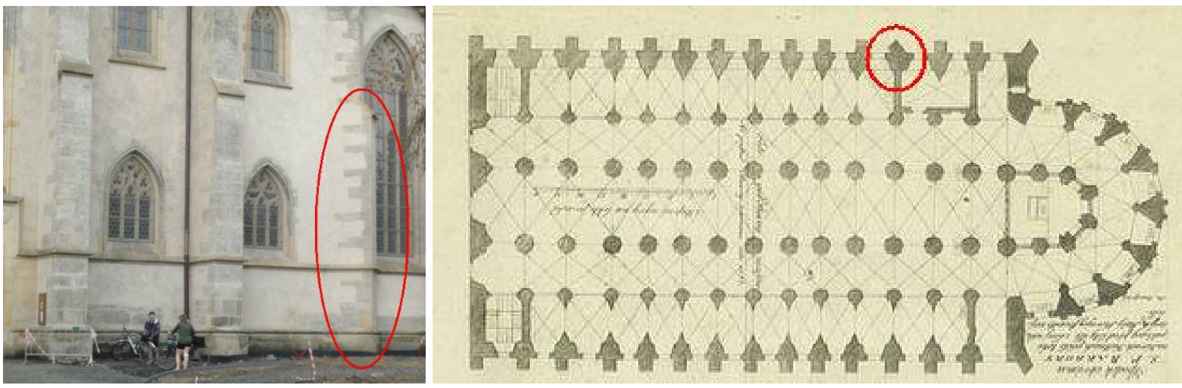

Figure 24 Different shape in column a) elevation b) plant [19]

From the outside three different types of buttress can be distinguish in correspondence with the presbytery, the transept and the nave as can be seen in Figure 25. In all the cases where two level of flying arches are present, it can be seen that the lower arches have a smaller section than the upper ones.

Unfortunately, by the time of the visit the scaffoldings were already removed and it was not possible to see the buttress from a near point of view. However they seem to be in a good condition. In the buttresses that have been restored small dark spots can be seen. They are located in the places were the holes were made and then filled with lime mortar. [Figure 26]

Figure 25 Type of Buttresses

Figure 26 Strengthened flying arches

The access to the interior building is located on the north transept. The first interior part of the church that was inspected was the floor. There are some spots which reflect variations in temperature and moisture. Those differences appear due to the presence of various tunnels and crypts that go through the cathedral subsoil.

Figure 27 Spots on the floor

The foundation cannot be seen, however it is known that they do not have a constant thickness since the depth of the base adapts to the surface of the rocky soil. Historical documents do not make references to foundations repairs. It is important to remark that the current restorations include also the studies of the foundations and crypts with georadar.

Trough the visit to the chapels around the presbytery, the presence of paints allusive to miners and their lifestyle is noticed which is unusual for a church. The paints are not complete since parts of them have been lost along the years. The last restoration of the paints was made in 1937. Figure 28 shows different interior paintings of the church. The lower levels of some interior paints and walls show layering and lost of cohesion between layers caused by the presence of moisture. [Figure 29]

Figure 28 Interior paintings

Figure 29 Damage on the interior walls

The presence of moisture seems to be due to rain water that was not well evacuated and remains near the base of the wall. This water can draw up by capillarity action into the masonry generating the humidity spots. To eliminate the rising damp it is necessary to control the localize water sources. Figure 30 shows the current works from the outside wall. A geotextile is being place between the soil and the masonry and then the space between pillars is being covered by a porous fill. The pipes and gutters have already been renovated.

Figure 30 Geotextile and porous fill

On the other hand, it is remarkable that the church has natural ventilation helping to prevent humidity problems in the interior. The air can pass from the exterior to the interior through vents that joint the exterior walls of the church (in correspondence with the chapels) to the floor of the ambulatory. The air can also move from the interior to the exterior along holes that are located in the vaults. The vents under the floor in the chapels were done in the year 1937. During the current restoration works the vents were cleaned and the wholes in the vaults were reopened. The main problem of this system is that people tend to close the ends of the vents because the fresh air makes them feel uncomfortable.

Figure 31 Grilles at the end of the vents a) from outside b) from inside

The ribbed vaults of Saint Barbara church show distinctive arrangements. In particularly it can be notice that for each part they have a different design. Figure 32 show on the left side the typical gothic cross vaults of the chapels, and on the right side the jumping vaults of the ambulatory. Those vaults are decorated with colorful paintings contrary to the light and uniform color of the cross vaults of the laterals nave and the jumping vaults of the aisles.

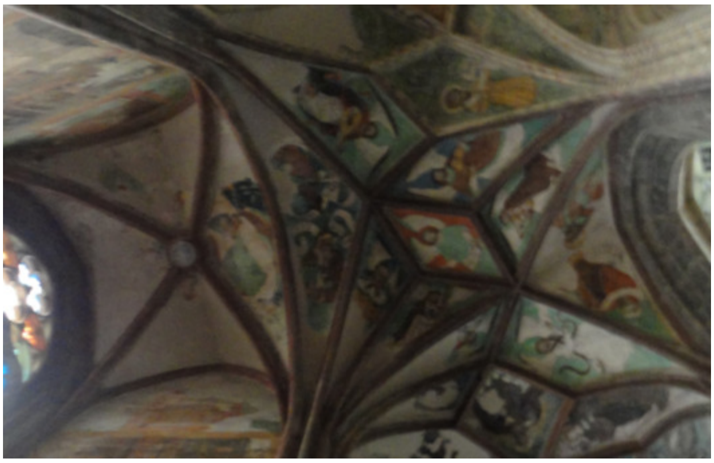

Figure 32 Vault of the ambulatory and chapels

Figure 33 a) Vault of the aisle b) Vault of the lateral naves

The jumping vaults of the presbytery are built from ribs that emerge from the pillar between windows to the lower arches and the thickness of the vault end in pointed arches in the windows. These vaults are covered from the inside with plasters. There are 39 shields on the surface dated from 1501.The vaults of Rejsek were restored during 2007 and at the present do not show damage. [Figure 34]

The vault of the presbytery is higher than the vaults of the principal nave; as a result there are not equilibrated horizontal forces in the joint between them. The necessity of an element sufficiently long and rigid in this joint was satisfied, during the restoration of the end of the 19th century, by a railway rail which still forms part of the building. [Figure 35]

Figure 34 a) Presbytery vault b) ribs emerging from the pillars

Figure 35 Railway rail located between the vaults

The jumping vault of the principal nave has a design compose by a combination of six pointed ribs in the central part and four pointed ribs in the vaults corresponding to the tribunes. On the vaults it can be seen humidity spots and detachment of the surface covers. [Figure 36] These decays were originated due to the bad state of the roof. After the restoration of the roof the water filtration was stopped. However the vaults were not restored jet.

Figure 36 Decay of the vaults

Due to the façade was not built until the end of the 19th century, tensors bars bracing between the springing of the arches located in the vaults (at the upper level of the pillar) were built at the time of the construction of the vault. Those bars are still present in the structure and it can be notice that the part that was added later does not have them. The ribs of the vaulting never touch the bars in the columns, but penetrate into the space between them, highlighting the lightness of the structure as can be observe in Figure 37.

Figure 37 a) tensor bracing b) Joint between column and ribs vaults

The interior of the nave have slender pillars, some of them still have a bit of color from the old times. The current state of the columns is good and it can be notice that some stone were replaced in the restoration of the 19th century.

Figure 38 Interior columns

From both the interior and the exterior sides of the church it can be appreciate the current good state of the glazed windows. In the last years the windows were extracted and all the stained glasses and joint were restored. This restoration was implemented after concluding the mapping of the failures of the windows and the design of the repairs works.

During the inspection, places that are not accessible to the public were visit like the tribunes, the roof and the space between the vaults and roof.

In the visit of the tribunes, it can we notice that the windows from this level have transparent glasses. On the south side of the tribunes, a crack is present in correspondence with the construction joint between the gothic construction and the part that was added in the intervention of the 19th century. In order to monitor the crack, some fixed points are defined. Measurements are taken at least once a month. [Figure 39] In correspondence with this crack there are two smaller cracks on the north side.

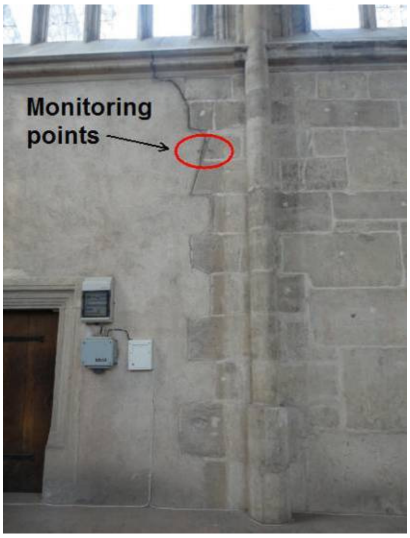

Figure 39 Location of crack

From the tribunes some parts of the triforium and the exterior passages that surround the roof of the chapels were accessed. Nowadays only four passages of the triforium are reachable because in 1884 some of them were walled up in order to increase the security of the pillars and its stability. [Figure 40]

Figure 40 Walls in the passages of the triforium

There are iron bracings along the windows and between the columns of the triforium. From the exterior it can be seen iron bracings at the level of the triforium and at a superior level.

The survey of the vault was also carried out from the space that is between the vaults and the timber structure of the roof. This area has a wooden platform with railing in order to allow people to access the place, mainly for maintenance proposes. The control panel of the heating system of the roof is placed there as well. It is possible to observe the profile of the vault from above because of the absence of infill over the vault. The vaults are made of bricks and are in good condition as can be seen in Figure 41 a).

Above the vaults there is a timber structure dating from the restoration on the 19th century. This timber roof supports the tent roof. The general impression is that the wood elements are globally in a good condition. In Figure 41 b) it can be seen that the central nave is connected to the lateral one trough timber tensors.

Figure 41 a) Extrados of the vaults b) Timber structure from the roof

The inspection also includes the roof of the church. It can easily notice that it is in a well state of conservation due to the restoration of 2002. The roof is characterized by its shape and its slate covering. A combination of different metals can be noticed on the outside of the roof, such as zinc and cooper. The last one is placed in the joints between metals and between metal and stones due to its good behavior against corrosion. Lead flashings are located along the cornices to protect the stone from water filtration. Stones sculptures with historical value are covered in its upper part with lead flashing too. Due to the malleability properties of lead, it is considered a very good material to covering irregular shapes.

Figure 42 Lead flashing on a cornice

The drain system is vital for the maintenance of Saint Barbara church. The nave's drain system is compose by gutters and pipes; while in the presbytery the upper flying arches from the buttress and gargoyles also take part of the drain system. The gargoyles and gutters on the roof have special sensors which turn on the gutter's and gargoyle's heating system when the presence of snow and frost is detected and automatically turn the system off when the snow is melted and the surface is dry. The gutters transport both the raining water from the roof and the water that is coming from the snow. [Figure 43]

Figure 43 Heating system a) On gargoyles with slate cover b) on gutters

Due to the high of the building, there are gothic fences along the perimeter of the three tents roof and along the roof of the presbytery to protect eventual workers from accidents. On the lower border of the three tents roof there is a wooden defense that protect the upper fence and the people from outside the church against suddenly snowfalls. Figure 44 shows the two levels of fences, there also can be seen the gargoyles of the presbytery and the wood defense of the main nave's roof.

Figure 44 Fences and wooden defense

8. TEMPERATURE AS A CASUSE OF DAMAGE [8] [10] [12]

Climate is considered one of the essentials causes of decay in buildings, breeding to failure of its materials and as a consequence sometime affects the structure. The resistance of materials against decay caused by climatic factors tends to decreases with age and their exposure.

Historical constructions, as well as any other kind of buildings, can be subjected to a large variety of climatic action during different moments of their time history. In particular, the active components of macroclimate that affect Saint Barbara church are radiation of the sun, seasonal and daily temperature changes and its combined effect with wind.

As a consequence of temperature variation, building material tends to expands when heated and contract when cooled, this expansion and contraction is called “thermal movement” and is a major cause of decay in buildings.

Due to the fact that thermal movements are not completely free, stresses are introduced in the structure. The stresses induced in building materials by temperature changes dependent on:

The properties of the materials involved as its elasticity, its capacity to creep or flow under load and its expansion coefficient.

The possibility of movements of the materials and the restriction caused by the connection with other elements of the structure.

The magnitude of absolute dimensional change in the material, which is calculated as the product of the dimensions of the material by the expansion coefficient and temperature differential.

The extent of thermal movement is influence by the temperature range. According to this, it is remarkable the fact that stone surfaces can reach temperatures much higher than the atmosphere.

The range of thermal movement varies according to the thermal mass. As mention before, the lower flying arches of Saint Barbara church are considerably smaller than the upper ones, consequently they have smaller thermal mass and are more sensitive to daily variations of temperature than the upper ones which have a higher thermal mass.

The location of the building is one of the factors that determinate its exposure to the climate. Saint Barbara church is located in a small hill, in an area where temperature can fluctuate significantly. In particular, the upper parts of the building are not shaded by its surroundings and are further away from the earth. Consequently the flying arches are subjected to greater solar gain and therefore tend to expand and contract more.

Masonry structures tend to adapt to cyclic thermal stresses by showing small cracks or deformations. As the expansion of masonry is not usually followed by equal contraction, small annual thermal movements can build up to large cracks over a period of years. In the arches of Saint Barbara church cracks tend to appear in the joints between stones on the flying arches. Consequently the contact surface between stones is reduced and the stresses in the stone increase. It could be said that the fatigue caused by repeated movements and the accumulation of deformations over the years are the main reason from decay in this flying arches.

From the structural point of view, the flying buttresses are weak points since they balance the deformation between the buttressing piers and the aisle wall. Consequently the expansion and contraction of the arches is also influenced by the behavior of the rest of the structure.

In order to be able to evaluate in a more proper way the influence of the temperature on the church under study, it was necessary to have values of temperature variation. During the years 2005 and 2006 temperature measurements were taken in selected points on the surface of the flying arches. Measurements were taken from one to twice a month. The aim of this previous study was to identify the difference in temperature between the lower and upper flying arches and the different behavior between the north and the south sides of the structure. In [10] a detailed explanation of the way in which the measurements were taken and the results are available.

As expected differences in temperature along the arches was found in the analysis, which bring a problem regarding the difference in elongation of the arches and as a consequence stresses. It was also noticed that the temperature in the lower arch change more quickly than in the upper arch as a result of the difference in areas. In Figure 45

Figure 45it can be clearly seen that the amplitude of the range of temperatures during a day is bigger in bottom arch than in the upper one. In addition the lower arch reaches the maximum value before than the upper arch. That is also a problem, since while the lower arch is already decreasing the temperature value; the upper arch goes on increasing its temperature.

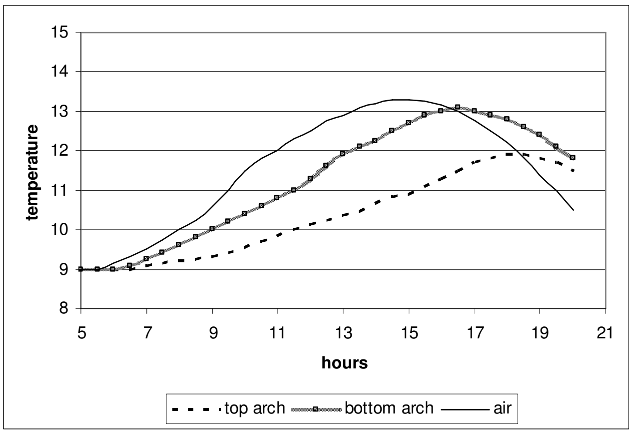

Figure 45 Temperature values at the central point of lower and upper arches 31/03/2005 [8]

In [8] the results of thermal analysis were used to define four load cases. Those load cases were used as thermal load cases in the model. It is important to highlight that they have remarkable difference of temperature simultaneously.

The orientation of the structure plays a fundamental role regarding the temperature influence. The South West part of the structure is much more affected than the North West part. This can be explained through Figure 46, where it is shown the temperature variation on a sunny day in the south and the north sides.

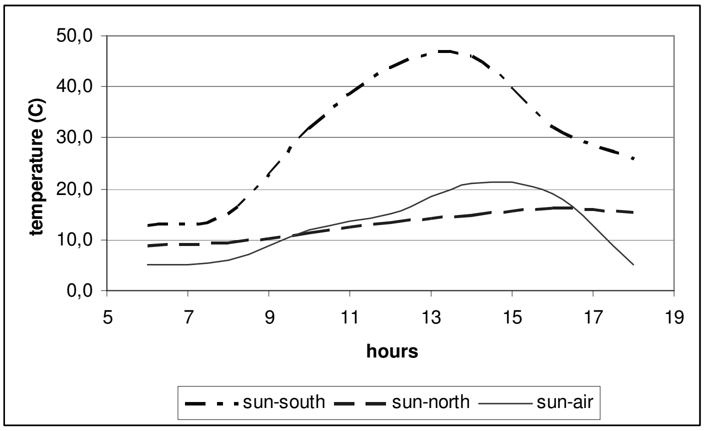

Figure 46 Temperature variation on south and north sides [8]

9. MATERIAL CHARACTERIZATION

The structure was made on masonry which is a heterogeneous material. It was originally created from limestone blocks and bounded by lime mortar. In the reconstruction of the late 19th century the material that was used for the changed and supplemented elements was sandstone. That explains the difference between the original and the reconstructed parts of the church.

Masonry is a composite material that is non-elastic, non-homogeneous, and anisotropic structural material, with different behavior in compression and tension, and low or negligible tensile strength. Mechanical characteristics depend on those of the constituent materials (units, mortar and their interfaces).

It has been recognized that the success of masonry analysis using finite element analysis largely depends on the careful selection of a suitable constitutive material model and the correct input of material constants associated with these models.

In previous studies material homogenization was used to analyze the linear as well as non-linear behavior of the structure.[8] In this thesis the focus was to model the structure describing the stone and the joints separately. To do this one material model for stones and other for joints were needed to be described. They require test data input to represent the non linear behavior accurately. However, performing the required tests to get all the necessary values is not always possible in historical constructions.

9.1 Stone

"Stone is a heterogeneous material which properties and durability are variable". [26] In order to characterize the sandstone for the fine element model some destructive tests on small non-standard specimens were performed. The samples were extracted from the building from the drilled holes made on the arches in the last restoration.

Unfortunately the size and number of samples available were not enough to follow the standard test. However they allow to have an idea of the behavior of the stone. The description of tensile and compression test performed are described below.

9.1.1 Tensile Test

The uniaxial direct tension tests were carried out using a universal testing machine (Model Alliance RT/30, MTS) under force control conditions at a rate of 0.05 mm/min. A 30KN load cell was used to measure the applied load. The testing software TestWorks 4 (MTS Systems Corporation) was used to control the testing machine and to acquire the instantaneous machine crosshead displacement and corresponding applied load data.

A direct tension fixture was needed to fix the specimen. The fixture has upper and lower spherical seats and leaf chains (oriented 90° from each other) for proper specimen alignment. With this fixture, the specimen was glued to end caps with an epoxy. When the epoxy has had time to set, the specimen end caps are attached to the chains with the provided clevis pins. Figure 47 shows the typical direct tension test configuration.

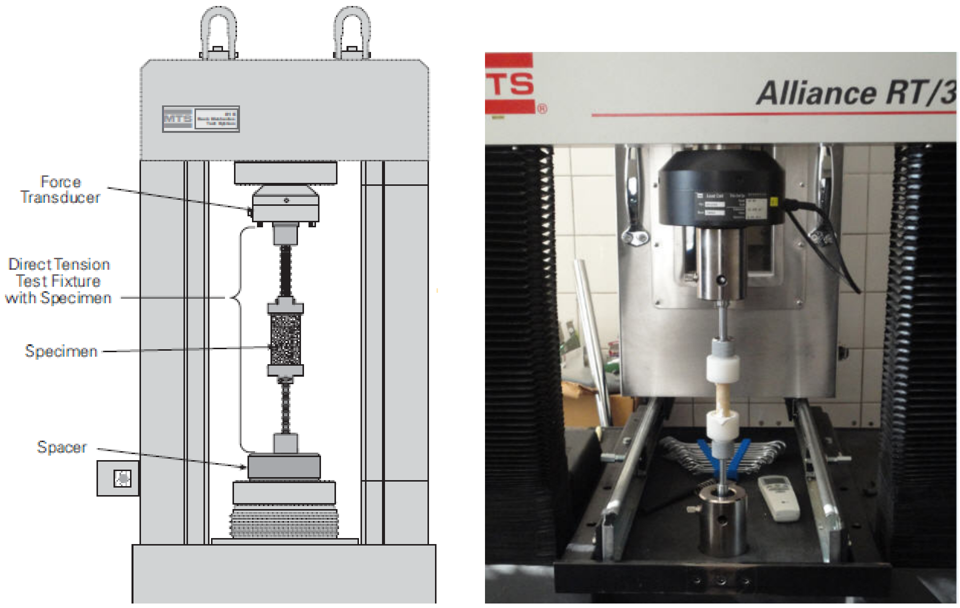

Figure 47 Typical direct tension configuration [18]

Six samples were tested. All of them were measured before the test. The load and crosshead displacements were recorded by the computer. After the test the tensile strength was calculated dividing the load by the cross section of the specimen. The average value of the tensile strength was selected to be used in the model ($\sigma_t = 1{,}22$ Mpa).

Table 1 resumes the geometry of the samples and results of the test. Figure 48 Show the Load – Displacements curve.

Especimen N°	Lenght (mm)	Diameter (mm)	Max Load (N)	Max Tensile Stress (Mpa)
1	45.0	17.5	322.01	1.34
2	45.0	15.0	296.65	1.68
3	45.0	17.4	208.96	0.88
4	46.5	17.2	261.98	1.13
5	61.0	17.7	237.18	0.96
6	65.0	17.6	331.15	1.36
		Average=	276.32	1.22

Table 1 Geometry and Results of Tensile Test

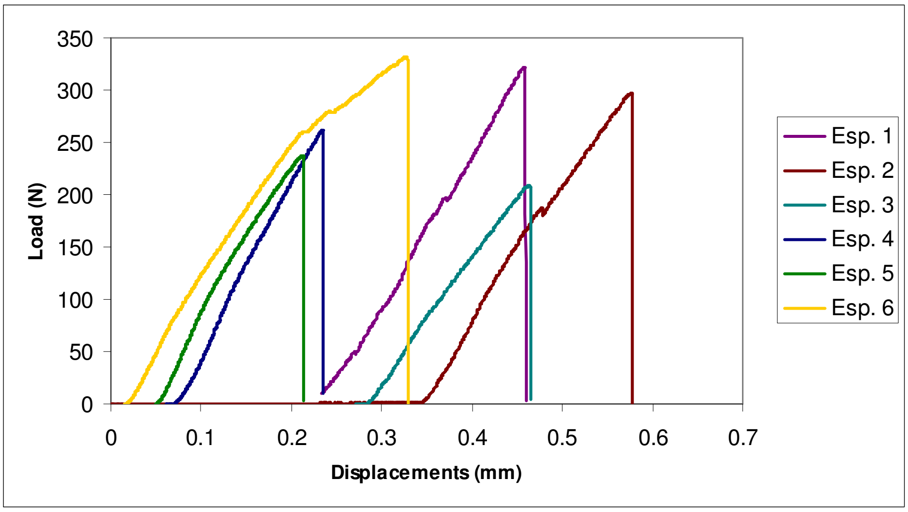

Figure 48 Load - Displacement curve in Tensile Test

The samples show a fragile fracture and a heterogeneous behavior as it can be observed in the load deformation curves. Figure 49 show sample N°3 broken. It can be seen that the fracture has a plane shape.

Figure 49 Fracture plane of Specimen N°3

The stress–strain curves (σ–ε) were also plotted and the area defined below the curve was measured in order to approximate the value of the fracture energy. However the results obtained were not representative due to the necessity of deduce the strain values since an extensometer was not used.

To obtain an accurate value of the fracture energy a three bending test should be performed. [26] However it was not possible because of the amount and size of the specimens available.

9.1.2 Compression Test

The uniaxial compression tests were carried out using the same testing machine used for the tensile tests (Model Alliance RT/30, MTS) under force control conditions at a rate of 0.08 mm/min. The testing software TestWorks 4 (MTS Systems Corporation) was used to control the testing machine and to acquire the instantaneous values of the signals like time, load, and deformation of whole specimen.

The compressive strength was tested on five samples, obtained after the tensile test, which was finished first. The samples were placed between two steel plates in such way that the lateral edges would be situated on the plate planes. This restricted the effect of the geometry imperfections on the sample surfaces, which had been cut off and sanded. A spherically seated upper platen was used to align of platen to the specimen surface. For the samples N° 3 to N°5 an extensometer was added to measure the strains. Due to the short length of the two first samples, it was not possible to use the extensometer on them.

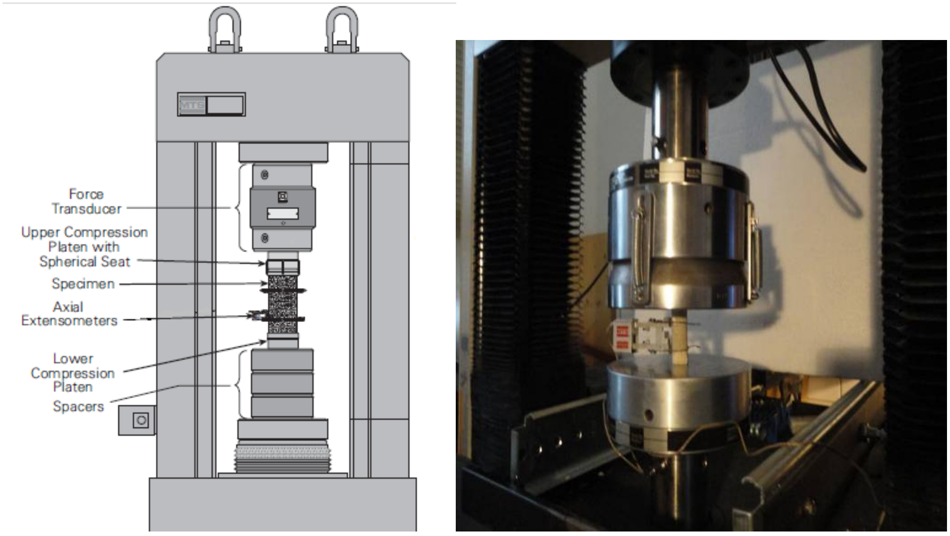

Figure 50 Typical Compression Test Configuration [18]

For the evaluation of the compression strength it was necessary to record signals like load and strain to the PC. Values of compression strength were calculated from this signal. The goal of measuring was to observe the compression strength evolution and to deduce a Modulus of Elasticity.

After placing the sample, the crosshead was moved to contact the specimen and generate the load specify by the Pre-load input. Then, the crosshead was moved at the speed defined before until the specimen reaches its endpoint.

Table 2 resumes the geometry of the samples and results of the test. It can be seen that the samples were heterogeneous in length, however the size effect do not influence much the maximum compression stress. Figure 51 Show the Load – Displacements curve.

Especimen N°	Lenght (mm)	Diameter (mm)	Max Load (N)	Max Comp. Stress (Mpa)
1	27.0	15.6	2500.6	13.08
2	27.0	17.3	2178.8	9.27
3	39.0	17.3	2409.6	10.25
4	53.7	17.5	3151.0	13.10
5	51.0	17.5	4141.1	17.22
		Average=	2876.22	12.58

Table 2 Geometry and Results of Compression Test

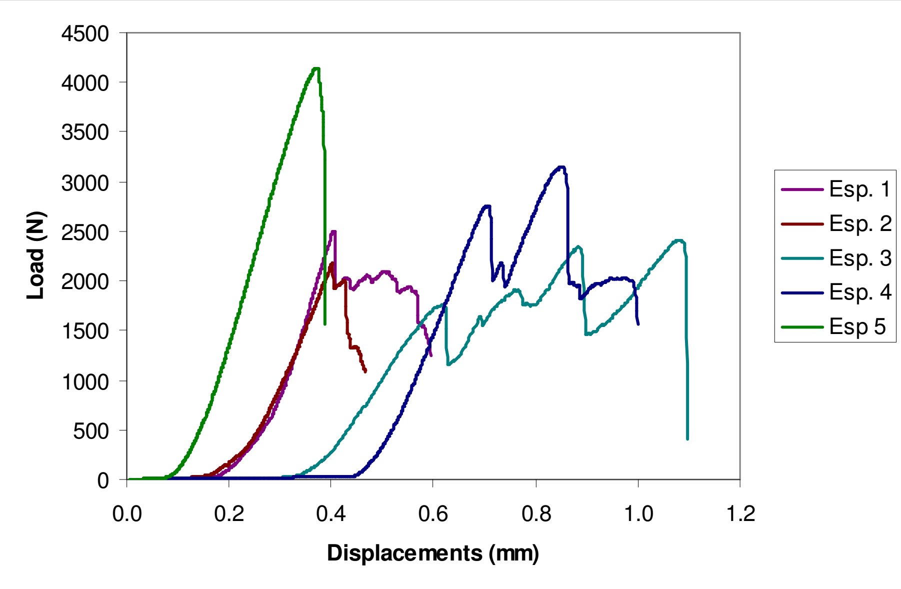

Figure 51 Load - Displacement curve Compression

Stress, which was calculated from load and loading area, and measured strain were used for the evaluation of the stress-strain diagram shown in Figure 52.

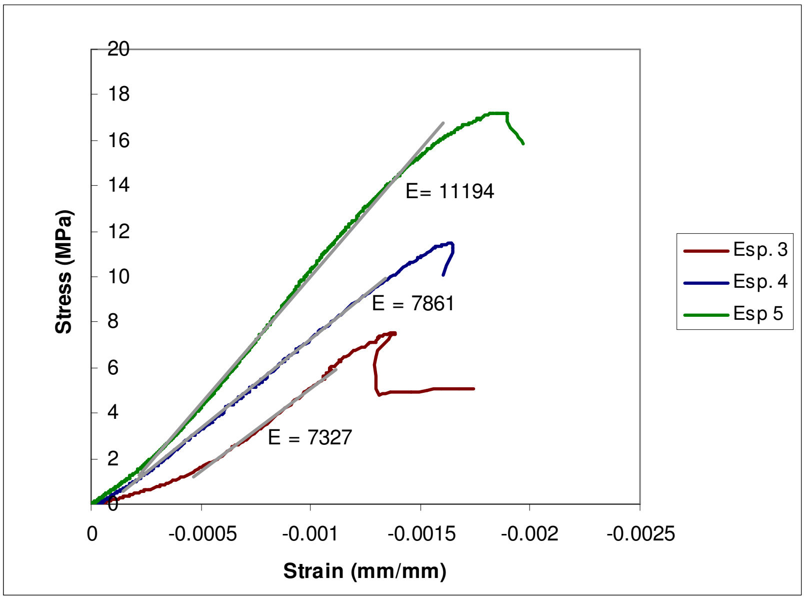

Figure 52 Stress - Strain curve Compression

From this figure the compression strain corresponding with the maximum compression stress was defined as $\varepsilon_c = 0{,}0014$. The average value of the compression strength was selected to be used in the model ($\sigma_c = -12{,}58\,\text{Mpa}$).

The Young's Modulus of Elasticity was calculated from the stress-strain curve. As can be observed in Figure 52 the values of each specimen has a pronounce dispersion. For the model the smallest one, corresponding with sample N°3, was used ($E_c = 7\,327\,\text{Mpa}$) since it is similar in value with sample N°4. Specimen N°5 shows a very good behavior that seems not to be representative of the whole structure. The value for the modulus of elasticity may be considered small; however, these results have to be taken with some reserves, mainly because the heterogeneity of the material.

From the test it can be said that the samples had a good compression behavior characterize by well define longitudinal planes of fracture. Generally the specimens showed longitudinal cracks along their longitudinal axes before the final fracture. Pictures of samples broken are shown in Figure 53.

Figure 53 Samples 1 to 5 after Compression Test

9.2 Joints

Unfortunately it was not possible to perform test on the mortar and their properties should be base on previous studies. A complete description of the properties used for the design is available in 11.1.2 Definition of materials.

In order to describe the behavior of the joints between the stone, shear tests should be carried out on the lime mortar interface. The shear strength of masonry joints should be evaluated by monitoring the sliding along their discontinuity plan. Tangential displacements should be imposed under a given constant normal force, and both the corresponding tangential forces and the normal displacements should be measured.

10. STRUCTURAL ANALYSIS

The structural concept of this cathedral is a rational combination of elements which allows a structural functionalism, which can be attributing to its gothic style. The structural analysis can be divided in the analysis in the transversal and longitudinal direction.

Regarding the transversal direction, the structural analysis is characterized by the following structural elements of the church: ribbed vaults, columns, piers and buttresses.

The ribbed vaults of the central part of principal naive conduct its weight and verticals load to the interior columns of the arcade. While the ribbed vaults of the aisle conduct its weight to the interior columns of the arcade and also through the supporting arches to the buttresses on the perimeter of the building. Due to the distribution of forces in the buttresses, the walls do not have a load-bearing function and as a consequence they are light and the design of huge windows was possible. A simplify free body diagram of the structure is shown in Figure 54.

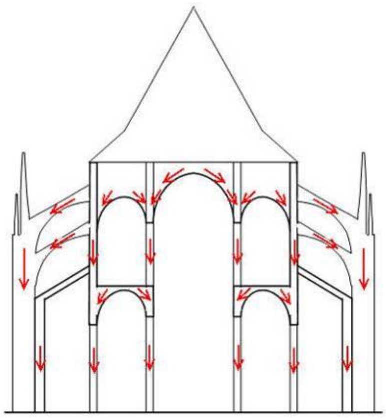

Figure 54 Free body diagram

Certain attention should be paid to the buttress since there are vital for the stability of the structure. They can be dividing in a massive vertical block, usually called pier, and the flying arches. The former transmit the load to the ground while the latter, diffuse the forces from the wall to the piers. Pinnacles and statues are placed in the upper part of the arches adding to the downward weight, and counteracting the outward thrust of the vault and buttress arch as well as stress from wind loading.

Because of its location, the lower arches transmit the loads in a more proper way from the wall to the buttress than the upper arches. However the inferior arches have a considerably minor section than the superior arches. According to previous studies the lower arches carried out approximately the 85% of the load while the upper ones around the 15%. [8]

The structural analysis along the longitudinal direction is characterized by tensors that stabilize the structure against the horizontal thrust forces. At the springing of the arches located in the vaults in the longitudinal direction, there are tensors for longitudinal bracing since for a long time the façade was not build and the arches needed a secure in the longitudinal direction. Also along the windows of the triforium and the laterals ones it can be seen steel tensors to work against wind forces.

11. MODELING

The modeling phase was a vital step in the development of this thesis since it allows a better understanding of the structure. The software used to carried out the model was ATENA 2D.

The model consists of the south buttress of the church façade [Figure 55]. This buttress was selected to perform the analysis because of its particular situation of lack or neglected pre stress coming from the weight of the vault. This particular condition allows the development of joint opening easier.

Using an existing CAD drawing, the joint and lines that create the model were defined in AutoCAD and then were imported to ATENA. The rest of the input data for the program, such as the generation of the macroelements, definition of loads, materials and supports, were prepared through the ATENA 2-D graphical preprocessing window.

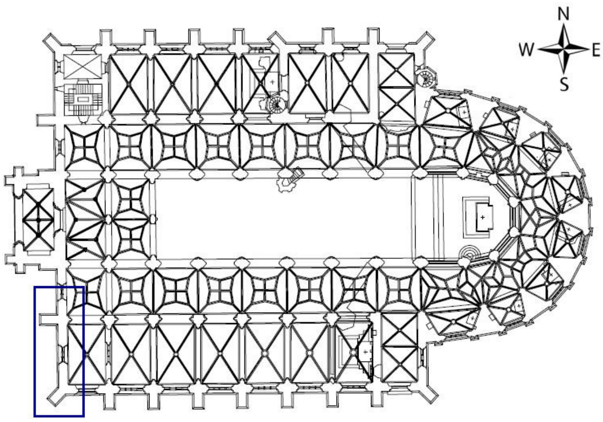

Figure 55 Location of buttress under analysis a) plane b) view

11.1 Model 1: Study of the original system

This model was carried out to analyze the behavior of the structure before its strengthening and the effect of temperature on it. Trough the comparison between the results and the recorded damaged present at the structure before the intervention, the model was validated and was able to be the base of the strengthened model (Model 2).

11.1.1 Geometry

The model consists of 455 joints, 765 lines and 310 macroelements. There are two types of connections between the macroelements: rigid and interface. The former is used when both macroelements belong to the same block of stone and the latter in the cases were the macroelements form part of different blocks of stones. In these cases the interface connections represented the lime mortar joint. In that way it was possible to define in the model each block of stone and its corresponding thickness. The analyzed buttress geometry is shown in Figure 56 a.

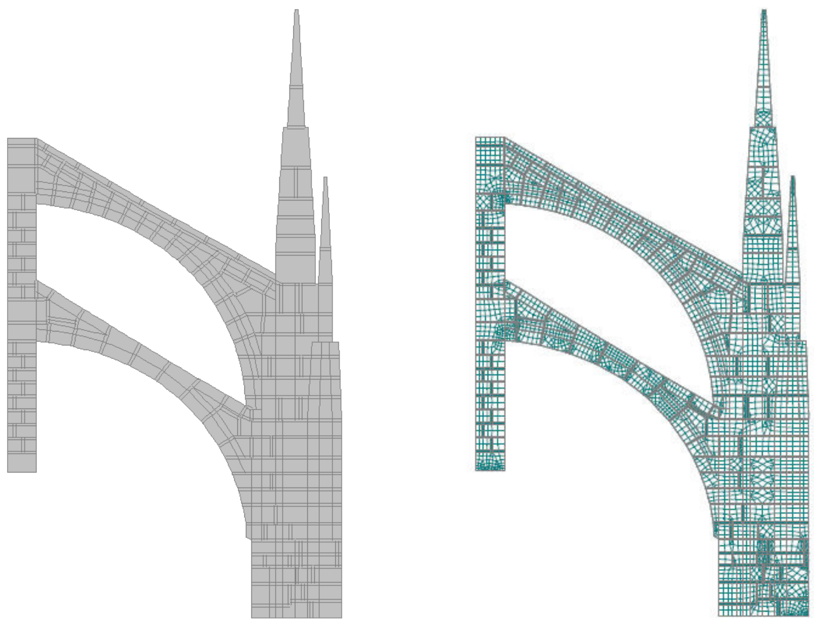

Figure 56 Geometry Model 1 a) Macroelements and joints b) Mesh

The macroelements were discretized by a mesh with mixed elements type. To ensure good accuracy of the analysis, the element size was specified at 0, 15 m. Where necessary a mesh refinement was defined. (Figure 56 b)

The thickness of each macroelement was defined following this rule: the transversal area of the macroelement should be the same as the area of its corresponding real section.

The geometrical limits of the model were defined considering the influence of the different structural elements in the buttress under study. According to this the façade wall was not modeled, however its effect was represented by support conditions. On the other hand, the thickness of the aisle wall was included in the model since the interfaces between arches and wall are places where damage was present. The vaults were not included in the model since the horizontal forces of the vault are transferred to the west wall, not having influence in the behavior of the buttress under study [8].

11.1.2 Definition of materials

For this model three types of materials were defined. One for the stone macroelements, other for the lime mortar joints and other for the springs supports which represent the masonry wall.

Stone

The mechanical parameters of the stone were based in laboratory tests, described previously, and results of preceding studies on the same kind of materials.

Stone was modeled employing "Sbeta material" with modified mechanical properties. Relevant values of them are given in Table 3.

Elastic modulus, E [(1)]	7.30E+03	Mpa
Poisson's ratio,	0.2	
Tensile strength, f_t [(1)]	1.22E+00	Mpa
Compressive strength, f_c[(1)]	-1.26E+01	Mpa
Specific fracture energy, G_f	5.00E-05	MN/m
Compressive strain at compressive strength in the uniaxial compressive test, ε_c[(1)]	-1.40E-03	
Reduction of compressive strength due to cracks	0.8	
Critical compression displacement, w_d	-5.00E-04	m
Specific material weight, δ	2.30E+03	MN/m^3
Coefficient of thermal expansion, α	1.00E-05	1/°C

Note: [(1)] Values deduce from laboratory test

Table 3 Characteristic values of Stone model material

The non linear stress strain diagram defined for this material is shown in Figure 57. The material state numbers that are shown in this figure indicate the state of damage of stone.

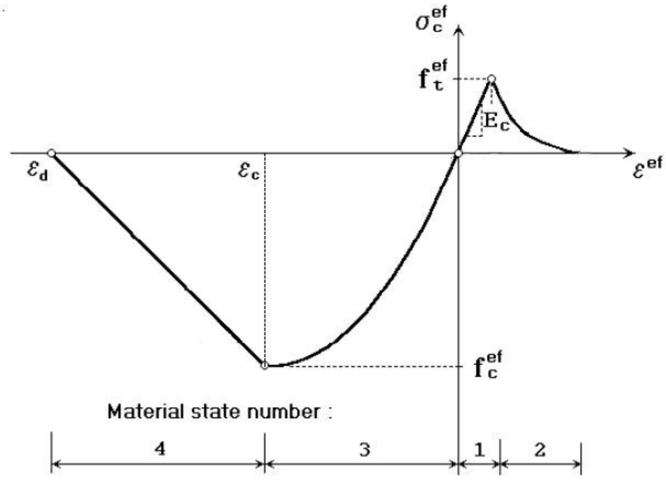

Figure 57 Stress-strain law for stone [5]

State number 1 represents the stone in tension before cracking. This behavior is assumed linear elastic, where E_c is the initial elastic modulus of stone and f_t^{ef} is the effective tensile strength. The stone after cracking is characterize in the second material state. Tension is described in this state by an exponential softening curve and the value of fracture energy G_f.

The stone behavior under compression is defined by material states 3 and 4. The former describe the stone behavior before the Peak Stress. The hardening branch of this state is defined by the formula recommended by CEB-FIP Model Code 90 [Figure 58]. The later define the stone behavior after the Peak Stress by a linear softening law. This law is governed by the value of the plastic displacement w_d, which indirectly defines the energy needed for generation of a unit area of the failure plane. The default value w_d =0,5mm was assumed.

$$\sigma_c^{ef} = f_c^{'ef} \frac{kx - x^2}{1+(k-2)x}, x = \frac{\varepsilon}{\varepsilon_c}, k = \frac{E_o}{E_c}$$

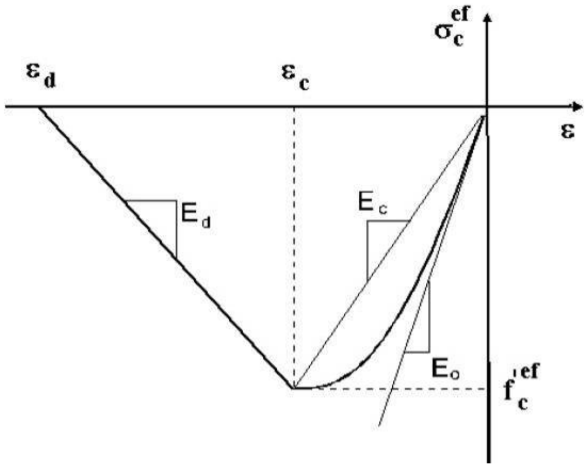

Figure 58 Compressive stress-strain diagram [5]

Lime mortar

The lime mortar joints between stones were modeled assigning a "2D interface material" to the connection lines. For each line it was necessary to define the thickness of the joint, which represent its deep.

The behavior of joint elements is controlled through the normal and tangential contact stresses between the two joint faces base on a Mohr-Coulomb frictional non-linear model with tension cut off. The failure criteria and it equations are show in Figure 59.

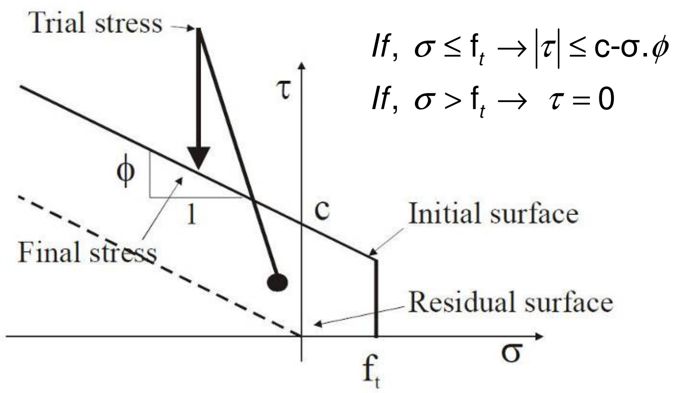

Figure 59 Failure criteria for interface material [5]

The parameters that were needed to be defined in the model can be categorized in two mains groups: the real physical properties of interface and stiffness coefficients, which serve purely for numerical purposes. The first group includes tensile strength f_t , shear cohesion c, and friction coefficient ϕ . The values of these parameters were taken from previous studies. [8] [4]

The second group contains the normal stiffness coefficients K_{nn} and the tangential stiffness coefficient K_{tt} . For those coefficient two values should be defined, the basic value which represent the rigid body and the minimum value that represent the open contact. Following ATENA manual recommendations, the basic value of stiffness coefficients were estimated using the following formulas:

$$K_{nn} = \frac{E}{t} , K_{tt} = \frac{G}{t}$$

Where t is the width of the interface zone and it was selected equal to 10 mm based on the average real thickness of the mortar between masonry bricks.

The minimal values, $K_{nn}^{\min}$ and $K_{tt}^{\min}$, are used only for numerical purposes within the nonlinear iterative solution method. The purpose of those values is to avoid the global stiffness to become infinite after the interface failure, because in theory, after the interface failure the interface stiffness should be zero. These minimal stiffness values were estimated about 0.001 times of the initial ones, according to the software manual. Significant values that characterize the interface material are given in Table 4.

Cohesion, c	3.50E-01	Mpa
Friction coefficient, ϕ	7.00E-01	
Tensile strength, f_t	5.00E-01	Mpa
Normal stiffness, K_{nn}	3.40E+05	MN/m^3
Tangential stiffness, K_{tt}	1.50E+05	MN/m^3
Minimal normal stiffness for numerical purposes, K_{nn}^{min}	2.00E+02	MN/m3
Minimal tangential stiffness for numerical purposes, K_{tt}^{min}	2.00E+02	MN/m3

Table 4 Characteristic values of Interface material

Spring

Spring material should be define to model the influence of the façade wall, which was represented as a support condition trough the assignment of springs on the bottom border lines of the model.

The spring material defines only the stress-strain relation in a spring, since its dimensions should be entered as properties of spring lines. The “spring elastic” type was used in this case and the spring stiffness, k, was defined equal to 7300 Mpa in correspondence with the Young’s modulus of stone.

11.1.3 Definition of Supports

Boundary conditions applied to the model shall represent those intended in the structure [8] and considering the effects of the limits of the models, which means the influence of the parts that are not modeled.

The effect of the facade wall is represented by vertical springs, along the bottom side of the pier and the aisle wall. Direction, orientation (Global in negative Y), material, width and length of the springs were defined through the spring option. Hence no additional vertical supports are necessary. In addition, the bottom right corner was fixed horizontally.

It was assumed that horizontal displacements along the vertical left border of the aisle wall were not possible to occur, due to the presence of the massive wall. Consequently, horizontal supports along the vertical line on the left side of the model were defined. Figure 60 shows the supports conditions.

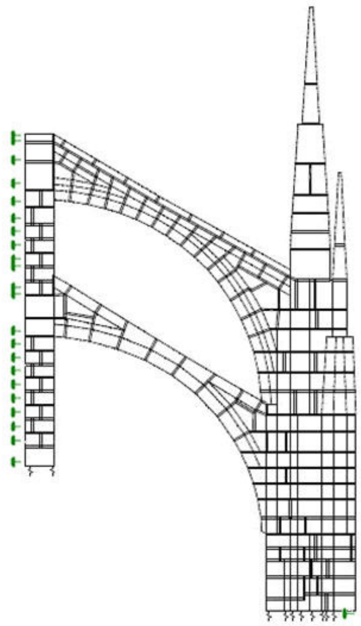

Figure 60 Supports conditions

11.1.4 Loads

The loads were chosen considering the previous studies [8] [10] and the Eurocode 1990 (Basis of structural design) recommendations [8]. The later classify the actions by their variation in time as permanent actions and variable actions. According to this classification the loads applied to the model can be described as follow:

Permanent actions

<u>Selfweight</u>

In this case the selfweight of the structure is the only permanent load consider. The self-weight was calculated by the program on the basis of the specified value of density (23 KN/m3). The load case "Body force" was defined by entering the component of direction vector (global direction –Y), no other data was required.

<u>Supports</u>

The support conditions were explained before. However, they were inputted into ATENA as load cases "Supports".

Variable loads

The effects of environmental influences were taken into account, considering the possibility of occurrence of the phenomena and its influence in the behavior of the buttress according to previous studies.

<u>Snow and wind</u>

Snow as a load itself can be neglected in this analysis due to the fact that the thickness and shape of the arches do not allow large accumulation of snow.

Because of the fact that the buttress under study is the one in correspondence with the main façade of the building, the transversal wall absorbs the effect of the wind load. Consequently the wind does not generate important stresses in this part of the structure and can be ignored in this analysis.

Additionally, the insignificant influence of these loads on the buttress was already proved. [10]

<u>Thermal actions</u>

As mentioned before temperature is an important cause of damage in Kutná Hora. It is known from previous studies that temperature has a dominate influence on the behavior of this structure. Thermal actions were determined according to existing studies. [8] [10]

Four different load cases were defined for temperature:

a) Minimum Temperature in winter: in this load case the lower arch has a temperature of -7℃, while the upper arch and the pier -3℃.

b) Maximum Temperature in winter: in this load case the lower arch has a temperature of -1℃, while the upper arch and the pier -3℃.

c) Maximum Temperature in summer: in this load case the lower arch has a temperature of 14℃, while the upper arch 8℃ and the pier 6℃.

d) Minimum Temperature in summer: in this load case the lower arch has a temperature of 3℃, while the upper arch 6℃ and the pier 3℃.

In all the cases temperature was applied to both macroelements and bar reinforcement trough a "Temperature" load case.

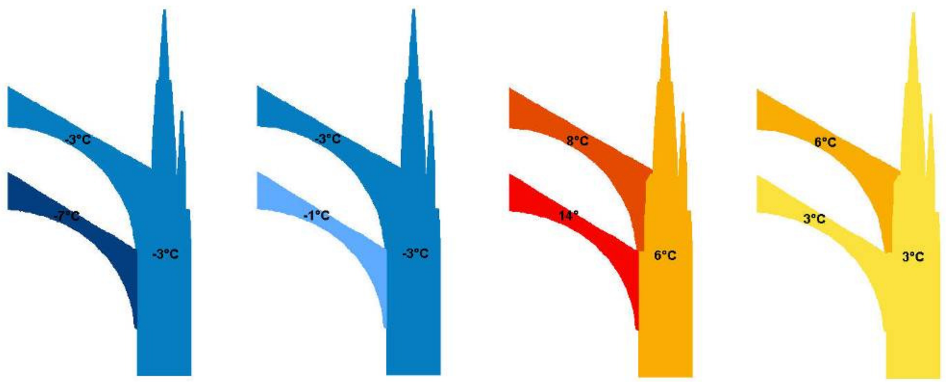

Figure 61 Temperature load cases

Due to the large thermal inertia of the nave, its temperature was considered for all the cases as in the initial state, which is the reference value defined as T=0℃.

11.1.5 Analysis steps and solution parameters

“Analysis steps” define the sequence in which loads are added to the model. In this point it is essential to remark that the analysis is non linear and as a consequence the load history influence on the results. As well, combination of actions is not possible. Therefore, to analyze the structure with different loads, the model should be run again with new load steps.

For each temperature load case a load history consisting of 60 load steps was defined. Where the first 20 analysis steps consist of the selfweight and the supports; and the next 40 steps, steps 21 to 60, add to the structure the corresponding temperature load. Each of the first 20 analysis steps represents 5% of the selfweight and the supports, while each of the subsequent 40 analysis steps corresponds to 2,5% of the temperature load. In order to show the results for each case, a name was assign to each load history:

a) Load history A: selfweight + supports + minimum temperature in winter.

b) Load history B: selfweight + supports + maximum temperature in winter.

c) Load history C: selfweight + supports + maximum temperature in summer

d) Load history D: selfweight + supports + minimum temperature in summer.

It is important to notice, that load steps in ATENA are incremental, which means that the load intensity in a current load step is added to the previously applied loads.

In all the load cases an amplification factor equal to one was considered, since the work was carried out in a verification phase, not a design one.

For each load step it was necessary to define its solution parameters, which define the solution method that are to be used during the non linear analysis. ATENA 2D contains a standard set of solution parameters, from them Standard Newton-Raphson solution method was employed due to its better stability.

11.1.6 Monitoring points

Monitoring points in significant locations were added in order to be able to follow the displacement thought the load histories. The monitored data provided important information about the state of the structure. In addition, it made possible to distinguish the step where the developments of joint openings took place. In each of the points shown in Figure 62 the horizontal and vertical displacements were recorded for each load step of each load history.

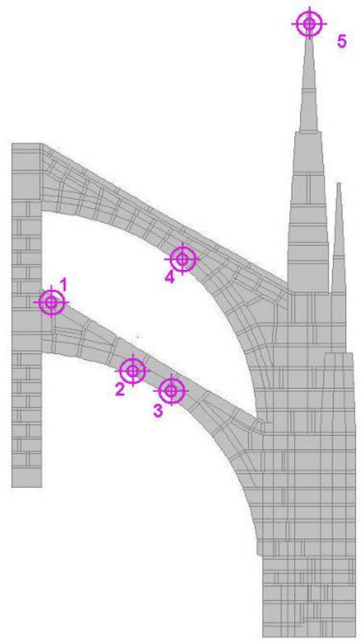

Figure 62 Location of monitoring points

11.1.7 Results

The most important outputs of the model are shown in this chapter. Because of the aim of the study, the deformed shape of the structure under temperature load is considered an essential result.

Figure 63 to Figure 66 show the deformed shape of the structure at the last step of each load history defined before, including the horizontal and vertical displacements, called X1 and X2 respectively. In addition, the maximums horizontal and vertical displacements are mentioned in each Figure.

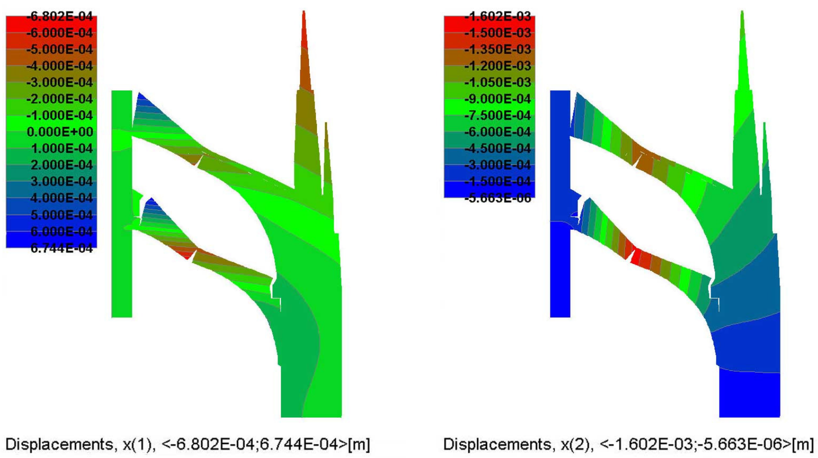

Figure 63 Displacements Model 1 Load history A

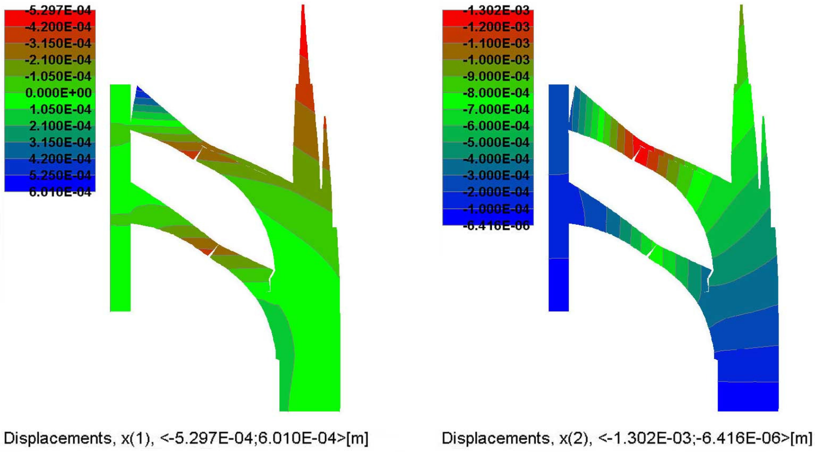

Figure 64 Displacements Model 1 Load history B

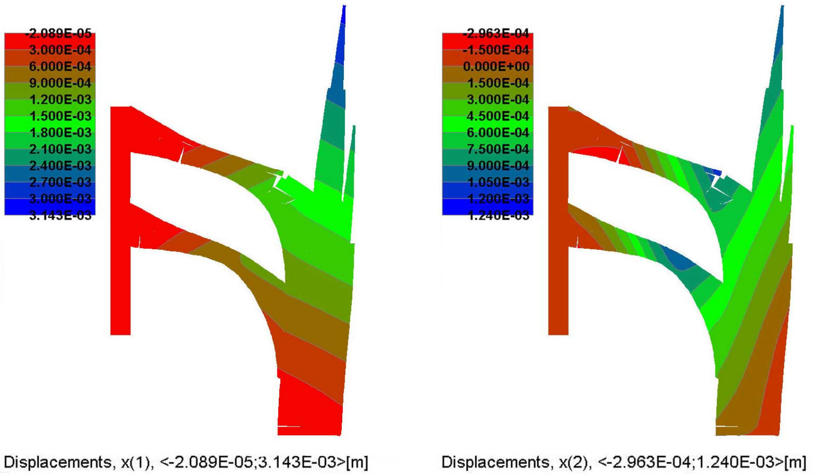

Figure 65 Displacements Model 1 Load history C

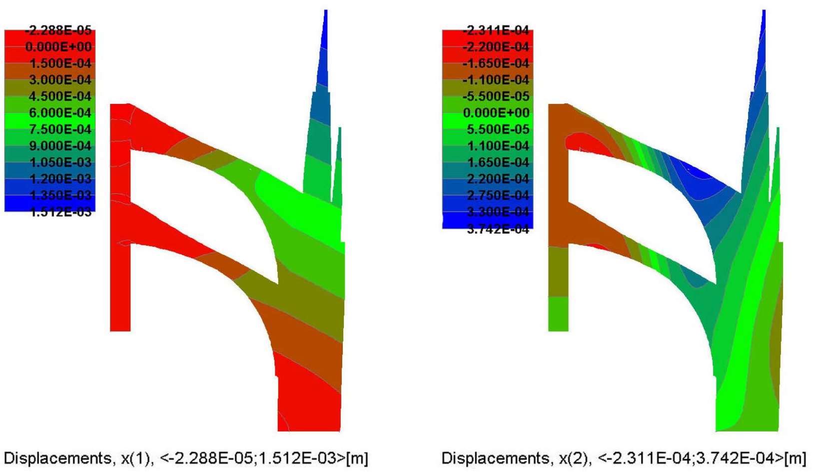

Figure 66 Displacements Model 1 Load history D

As expected the structure deform more in the extreme cases, it means minimum temperature in winter (Load history A) and maximum temperature in summer (Load history C). Regarding the shape of the deformed structures, it can be seen that Load history A have joint openings pattern similar as the one seen in the real structure. In particular, this Load history, show the three typical hinges that an arch develops when crack appears because of tension.

It can be appreciated that in Load histories A and B the central part of the lower arch tends to move down while in Load histories C and D it tends to move in the opposite direction, this movement after a certain number of cycles can generate fatigue and develop the failure of the structure. A further comparison from the movements of the monitoring points for each load history for Model 1 and 2 can be found in 11.3 Comparison between models .

It is remarkable that the values of the displacements in the model are smaller than in reality. However, the model is an idealization of the actual situation, which has a lot of approximations and hypothesis and consequently it cannot show the absolute real behavior. Also, it is notable the fact that in the model the load is apply in an hypothetically situation where not previous load history is considered in the structure neither previous deformations, what in fact is not true since the structure has already overpass uncountable cycles of temperature variations, both during the days and during the years. Moreover, after long period of thermal movements the mortar in the joint tends to loose its cohesion and consequently the material in the joint is missing. As a result the space between stones increases.

Since the principal aim of modeling is to compare the models it can be considered valid to use this model with smaller deformations values.

In order to understand better the behavior of the structure, its deformation was recorded in some significant load steps. The following figures show the evolution of the buttress trough the time history. (Figure 67 to Figure 70).

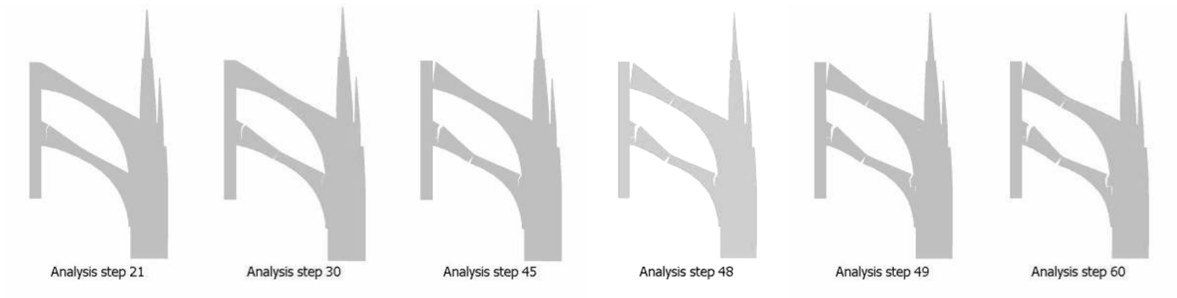

Figure 67 Development of deformation Model 1 (Load history A)

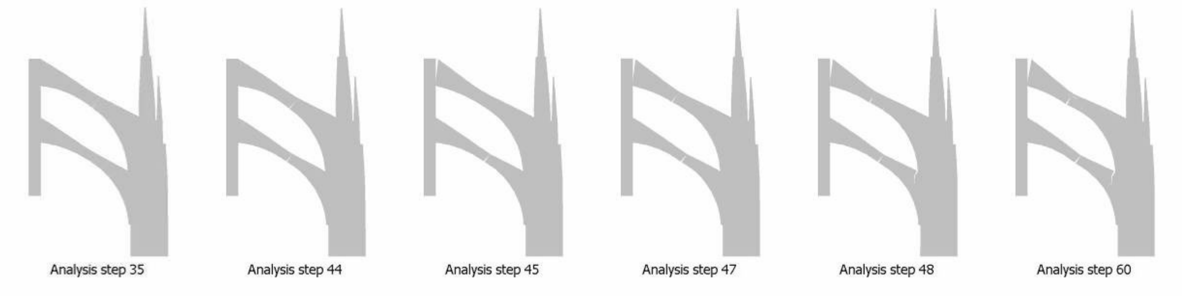

Figure 68 Development of deformation Model 1 (Load history B)

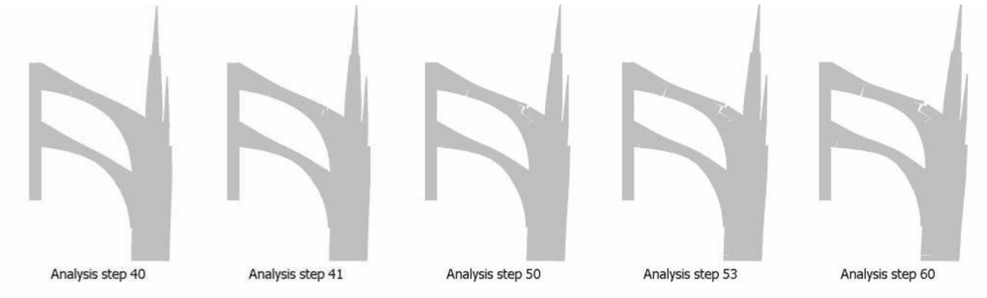

Figure 69 Development of deformation Model 1 (Load history C)

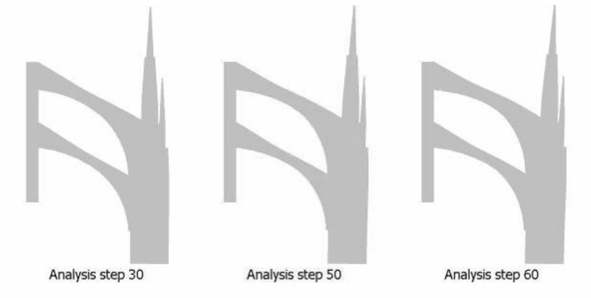

Figure 70 Development of deformation Model 1 (Load history D)

In Figure 70 it can be notice that the phenomenon of joints opening do not occur. That confirms the hypothesis that temperature in winter generates worst conditions for the structure than the temperature in summer.

Other significant parameters of the structure are principal stresses, their location and orientation. In the following pictures it is shown for step 60 the principal stresses and the tensor in the structure. (Figure 71 to Figure 74).

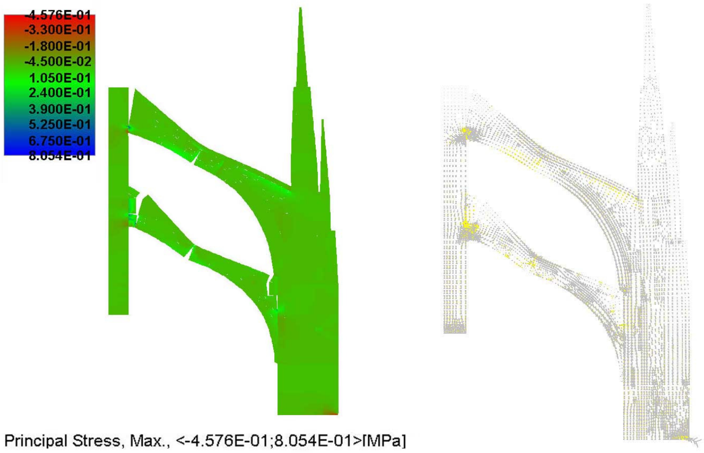

Figure 71 Model 1 Load history A a) Principal stresses b) Tensors

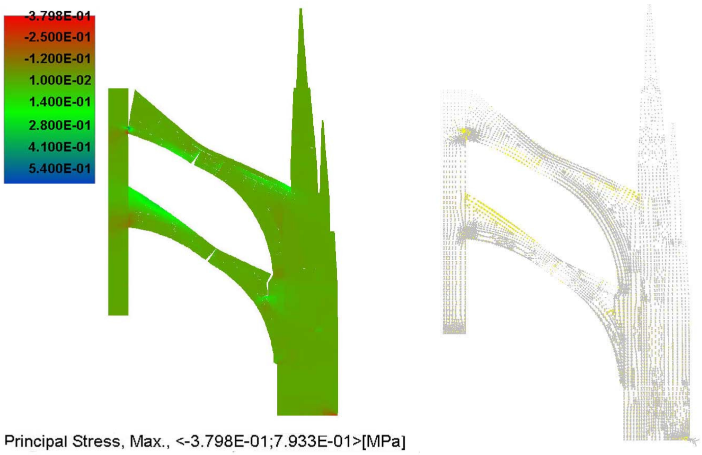

Figure 72 Model 1 Load history B a) Principal stresses b) Tensors

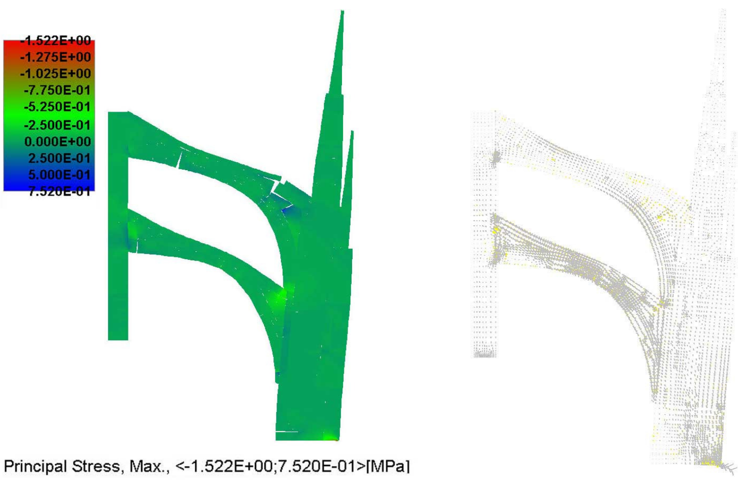

Figure 73 Model 1 Load history C a) Principal stresses b) Tensors

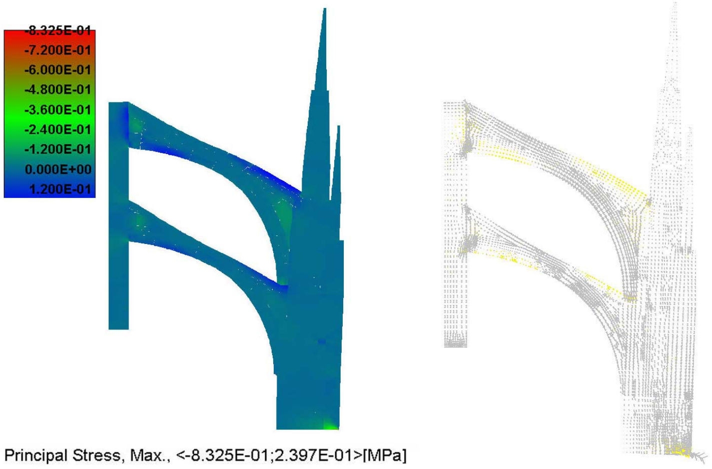

Figure 74 Model 1 Load history D a) Principal stresses b) Tensors

In the figures above it can be seen that the structure under Load histories A to C have similar values of maximum stresses, while under Load history D the maximum tensile stress is significantly slower. In all the cases it can be notice that the areas with traction are located in the places where the process of development of the cracks is taking place or is near to be started. The places where the crack is completely developed are characterized by compression forces. It can also be notice that the compression stresses predominates in the structure.

It is vital to mention that the structure do not present cracks in the stones in any of the four cases of load history. However, since the main problem has a cycle behavior cracks can appear after some temperature cycles.

11.2 Model 2: study of the buttress with the last intervention

This model was carried out to evaluate the behavior of the structure after its strengthening. In particular, the aim was to assess the influence of the reinforcement on the response of the structure under temperature loads. This model is based on Model 1 with the addition of the reinforcements which were added to the flying arches during the current restoration.

11.2.1 Geometry

The model consists of 455 joints, 765 lines and 310 macroelements and 28 bar reinforcements. The main difference between the geometry of this model and the previous one is the addition of the bar reinforcements, consequently all the explanation of the geometry in model 1 are valid in this model too.

The bar reinforcement were added as discrete reinforcements elements, their location was defined according to Cad drawings from the last intervention. The section of each bar reinforcement is the equivalent as two stainless bars of a diameter of 10mm, which they represent since there is a bar on each face of the arches. The analyzed reinforced buttress geometry is shown in Figure 75.

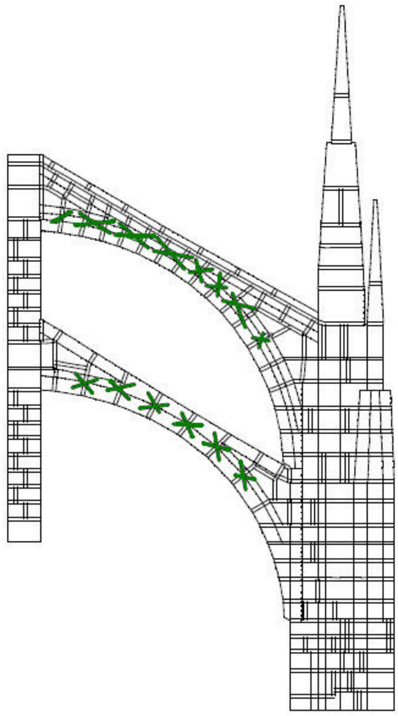

Figure 75 Geometry Model 2

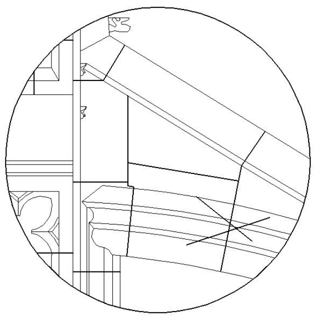

Figure 76 Detail of left voussoirs on the lower arch

Figure 76 shows a detail of the left side of the lower arch. There it can be notice that the last joint was not reinforced. Moreover, the especial geometry of the stones can be seen. This geometry was copy in the model, since it has a particular behavior regarding the possibility of sliding of the stones.

11.2.2 Definition of materials

For this model five types of materials were defined, the three types explained in model 1 and additionally two new materials for modeling the bar reinforcement and its bond with the stone.

Bar Reinforcement

The bar reinforcement material was defined through a bilinear stress-strain law, elastic-perfectly plastic, as shown in Figure 77. It can be characterized by an elastic part with a modulus of elasticity Es= 200 000 Mpa followed by a plastic part after reaching the yield stress ($\sigma_y = 500$ Mpa).

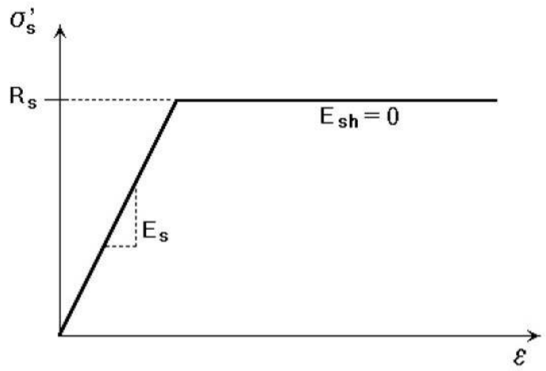

Figure 77 Reinforcement stress-strain law [5]

Bond of reinforcement

This material was used to represent the bond between the stone and the reinforcement. The basic property of this material is its bond-slip relationship, in this case due to the non availability of information, the model according to CEB-FIB model code 1990 provided by ATENA was used.

11.2.3 Definition of Supports

Identically boundary conditions defined in Model 1 were used in this model.

11.2.4 Loads

Since the main propose of this model is to compare the results with Model 1, the same load cases were applied to the structure. In particularly, in the temperature loads cases, temperature was applied also to the bar reinforcements.

11.2.5 Analysis steps and solution parameters

"Analysis steps" described in Model 1 are also valid for Model 2. Regarding the solution parameters, Standard Newton-Raphson solution method was employed as in Model 1.

11.2.6 Monitoring points

Following the aim of comparing results, identically locations of monitoring points were considered.

11.2.7 Results

As in model 1, the most important outputs of the model are shown in this chapter. Figure 78 to Figure 81

Figure 81show the deformed shape of the structure at the last step of each load history as defined before, including the horizontal and vertical displacements, called X1 and X2 respectably. Additionally, the maximums horizontal and vertical displacements were introduced.

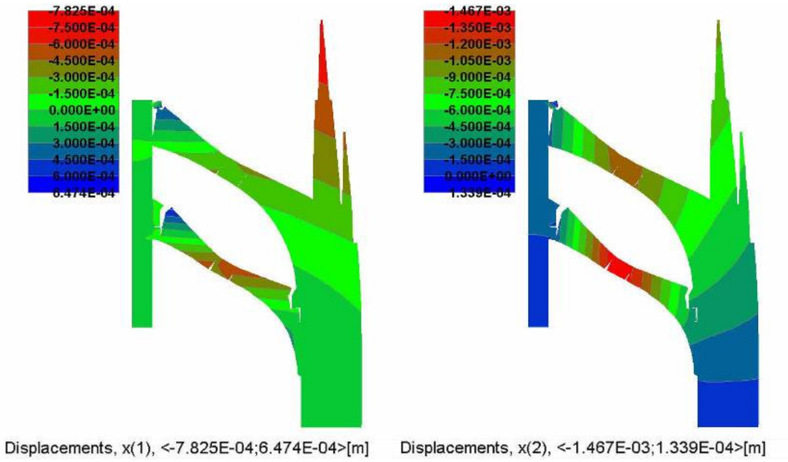

Figure 78 Displacements Model 2 Load history A

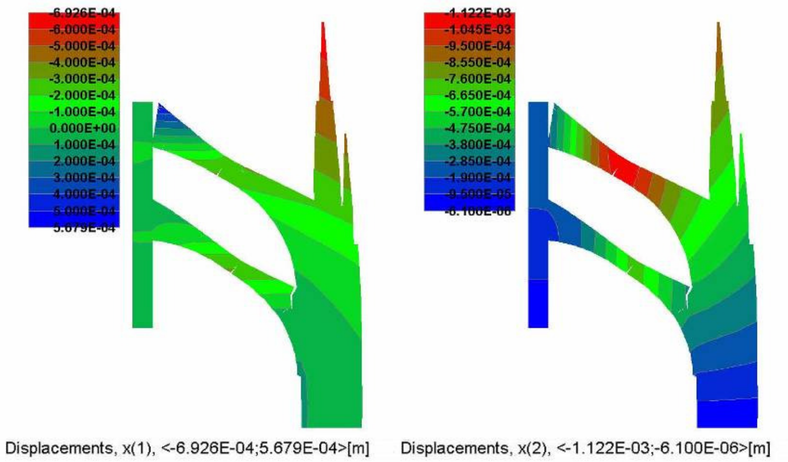

Figure 79 Displacements Model 2 Load history B

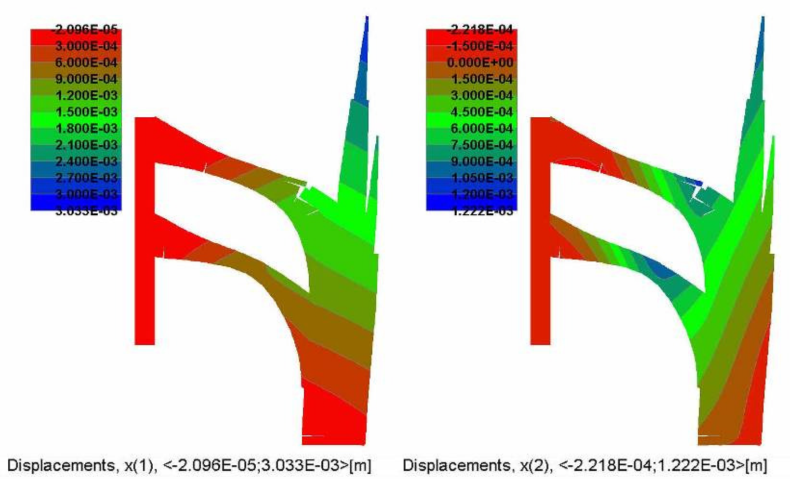

Figure 80 Displacements Model 2 Load history C

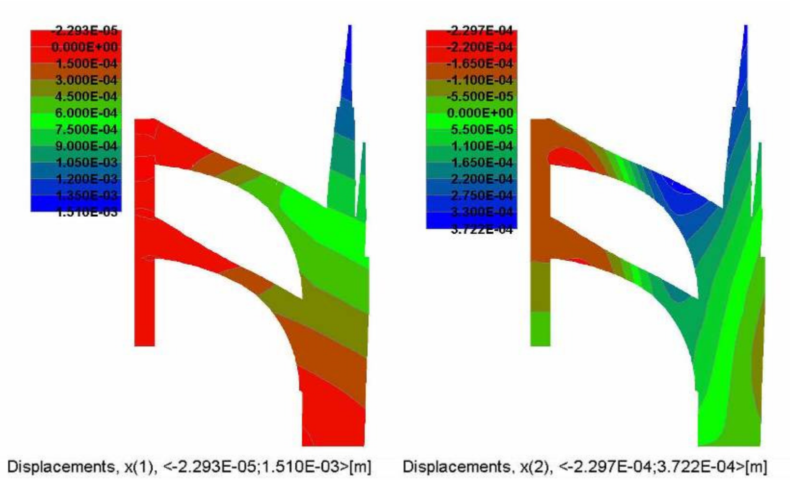

Figure 81 Displacements Model 2 Load history D

I would like to point out that in this model the structure deforms in each load history with a shape similar as in Model 1. However, the values of the displacements are smaller. In the load histories A and B the deformation of the lower arch is characterize by the opening of two cracks in the central part whereas in Model 1 only one crack was developed in that place.

The evolution of the deformation of the buttress was also recorded for this model. Figure 82 to Figure 85 show the evolution of the buttress trough the time history.

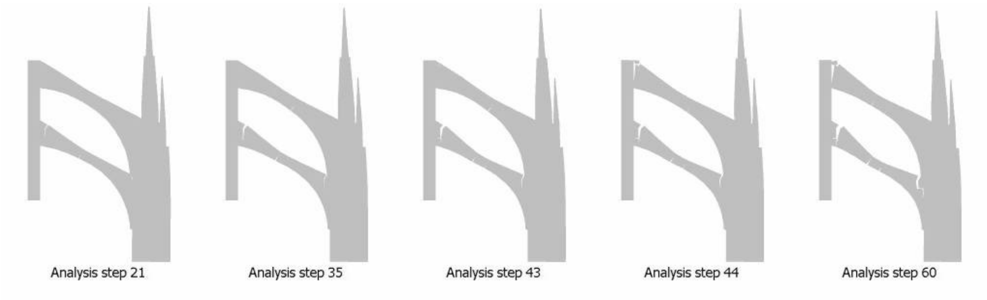

Figure 82 Development of deformation Model 2 (Load history A)

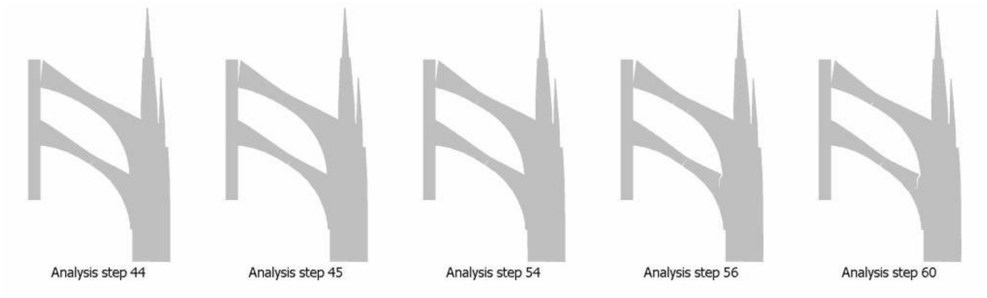

Figure 83 Development of deformation Model 2 (Load history B)

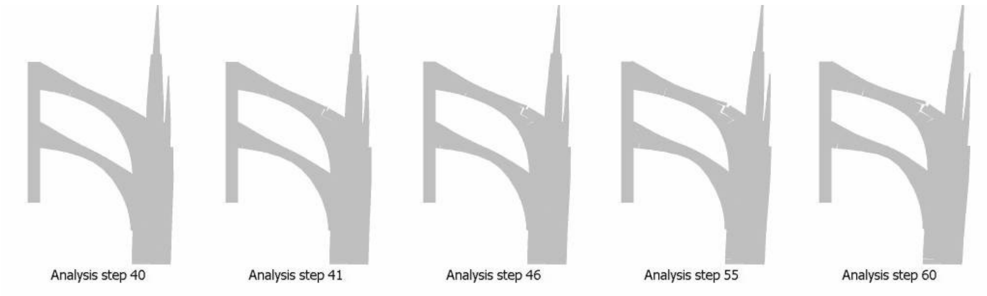

Figure 84 Development of deformation Model 2 (Load history C)

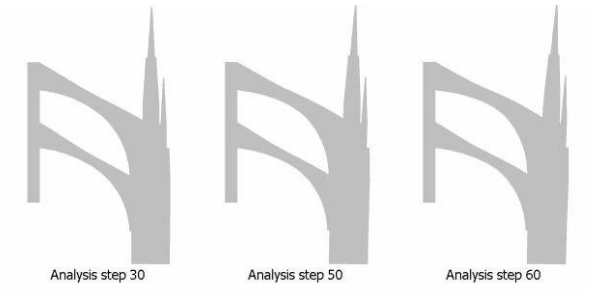

Figure 85 Development of deformation Model 2 (Load history D)

In the figures bellow it can be seen that the development of joint opening for load histories A and B occurs later than in model 1, which means that the reinforcements delay this phenomena. However for Load histories C and D, the reinforcements are not able to defer the development of joint opening because of the relative locations between reinforcements and joint openings.

As mention in Model 1, principal stresses, their location and orientation are considerable parameters of the structure. Principal stresses and tensor vectors in the structure for step 60 are shown in Figure 86 to Figure 89.

Figure 86 Model 2 Load history A a) Principal stresses b) Tensors

Figure 87 Model 2 Load history B a) Principal stresses b) Tensors

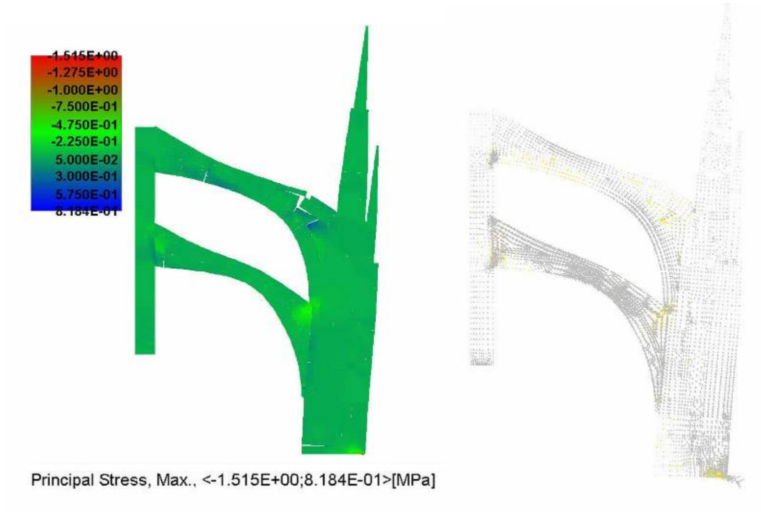

Figure 88 Model 2 Load history C a) Principal stresses b) Tensors

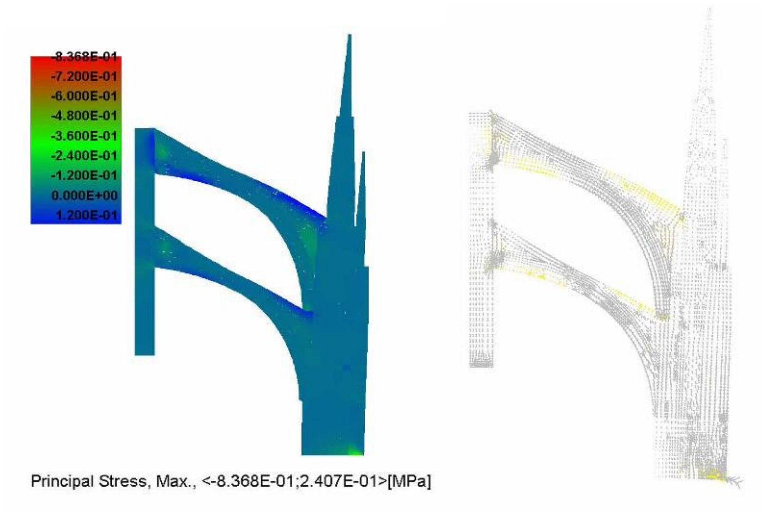

Figure 89 Model 2 Load history D a) Principal stresses b) Tensors

It can be seen that the values of principal stresses do not change significantly. However the distribution of stress is different since tractions are usually located near the places were opening of joints is not possible due to the presence of the reinforcement.

In this model it is also interesting to evaluate the stresses in the reinforcements. Figure 90 show the stresses along the principal axis in the reinforcements (stress sigma X). It can be seen that, as expected, the reinforcements of the lower arch have in load histories A and B higher stresses than the upper arch. While, for load histories C and D the reinforcements of the upper arch have higher stresses than the reinforcements of the lower arch. The maximum values of tension are small and far from reaching the yield stress ($\sigma_y = 500$ Mpa), that means that the reinforcements have more capacity to take eventually increases of tension.

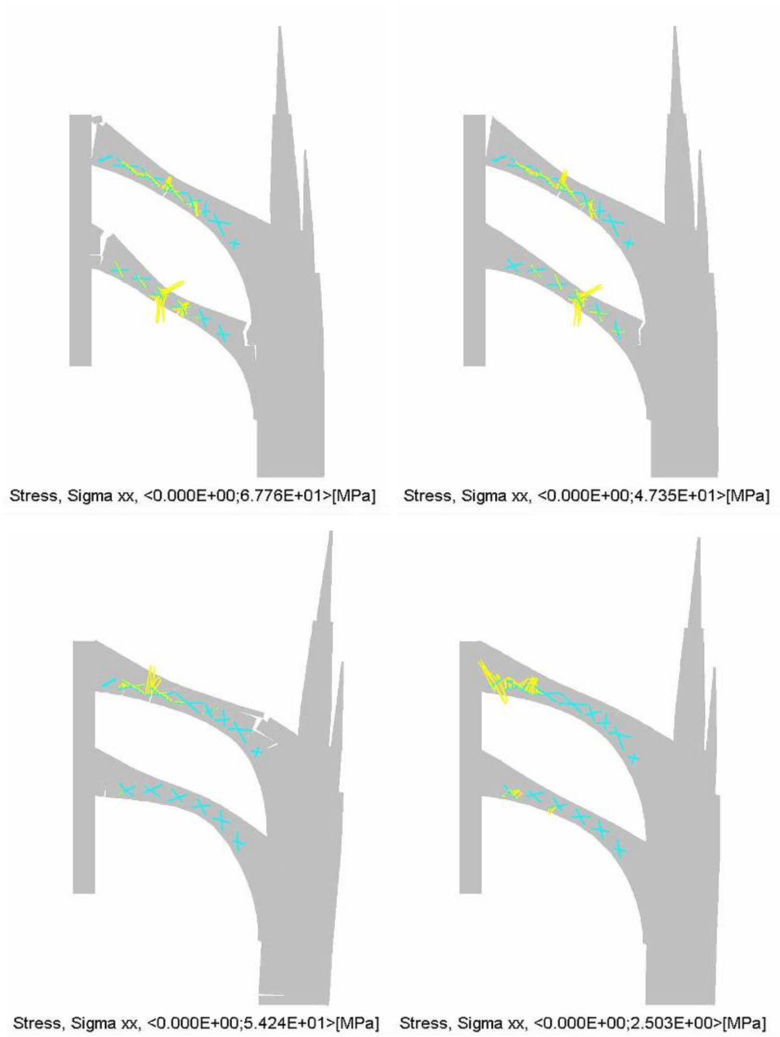

Figure 90 Stresses Sigma x in the reinforcement (Load histories A to D)

11.3 Comparison between models

In order to compare Model 1 and 2, graphics following the horizontal and vertical displacements of the monitoring point trough the load step were done. Those displacements are represented in the horizontal and vertical axes of the figures respectively. There are five figures, one for each monitoring point. In each figure there are eight curves, four of them representing the four Load histories for Model 1 and the other four corresponding to Model 2.

Above, Figure 91 shows the displacements for Monitoring point 1, which is located in the superior left edge of the lower arch. In the first 20 load step it can be clearly distinguish in all the curves a first branch due to self weight, which is exactly the same for all the cases. After this, for Load history A, horizontal displacements predominate. At load step 21 the joint located at this place opened, what can be appreciated in the horizontal step on the curve. It can also be seen that for Model 2 the final displacement is a bit smaller.

For Load history B, since the lower arch does not have a considerable change in temperature, the displacements are insignificant and similar for Model 1 and 2.

In the case of Load histories C and D the lower arch tends to have an expansion, which is reproduced in the figure as an ascending branch. This branch is practically linear since the point move without any opening of joints.

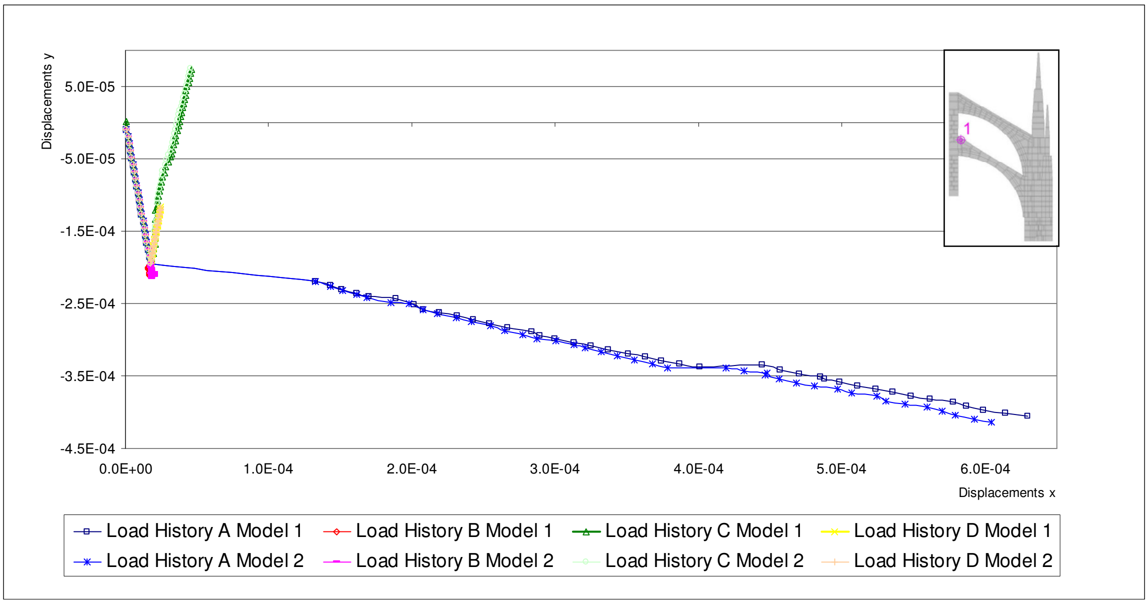

Figure 91 Monitoring point 1

In the central part of the lower arch two monitoring points (Monitoring points 2 and 3) were placed with the aim of supervise the joint opening phenomena for the different load histories. Monitoring point 2 is located at the center of the lower arch while monitoring point 3 is on the right side of monitoring point 2. Figure 92 and Figure 93 show the displacements for Monitoring point 2 and 3 respectably.

As for monitoring point 1, a first linear branch characterizing the first 20 loads steps can also be distinguish for monitoring points 2 and 3. After that, the curves that represent load histories C and D show an expansion of the arch, which is basically the same for the original system and the reinforce one. However in monitoring point 3 the points are align with a higher slope and consequently they reach a superior value of displacements.

In the cases of load history B, it can be seen that the reinforce model show smaller horizontal displacements. The value of the displacements are higher for monitoring point 3 than for monitoring point 2, which make sense since in this load history the opening of the joint take place in the position of monitoring point 3 and not in monitoring point 2.

For Load history A it can be observed that both monitoring points have the same displacement curve for Model 1, that is because only one opening of joint take place. The later is located in correspondence with monitoring point 2. However, the curves that represent Model 2 are different. The curve that show the displacements of monitoring point 3 under load history A for Model 2 has a higher value of displacements that the one for monitoring point 2, since joint opening phenomena occurs in both locations in that case. As expected, the displacements in Model 2 have a global behavior where the displacement of each point is dependent of the movements of the others due to the fact that the stones are joined by the rods. For instance, the displacements of monitoring points 2 and 3 are bigger for Model 2 than for Model 1 under Load history A, but it does not mean that the width of the opened joint is higher since the whole arch deform together and tends to move in the same direction. This situation of global movement can be also notice in Figure 95.

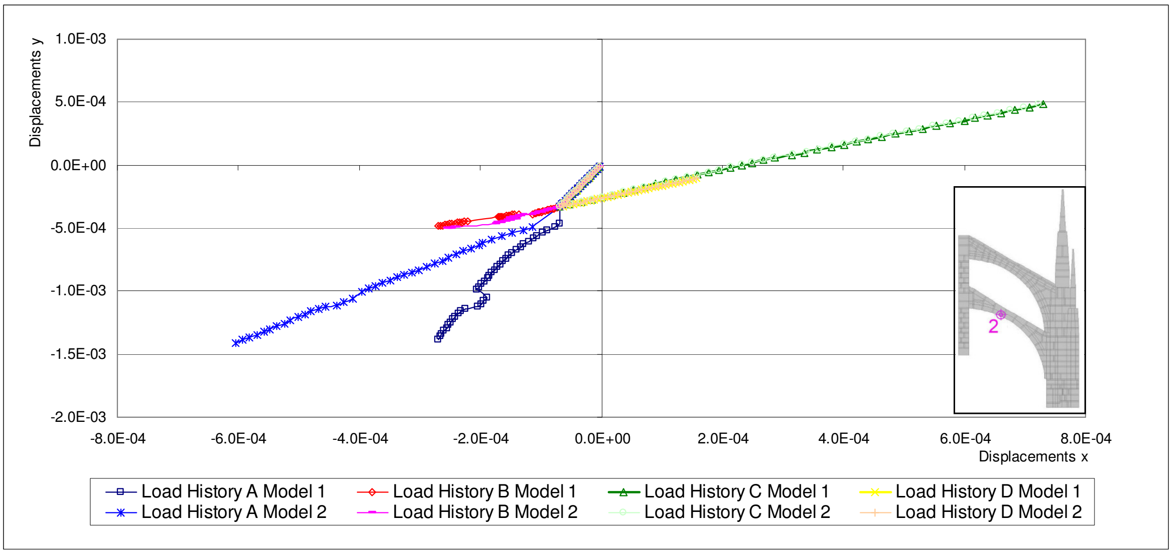

Figure 92 Monitoring point 2

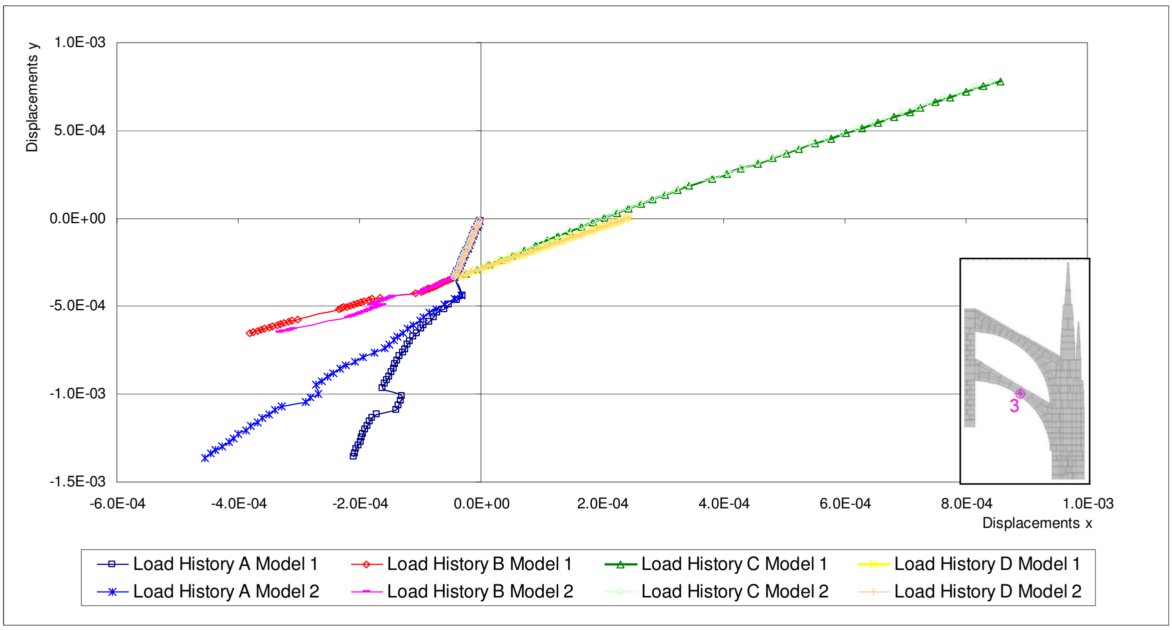

Figure 93 Monitoring point 3

In the following figure the recorded displacements of monitoring point 4 are shown. This point is located in the lower central part of the upper arch. In this case it is remarkable that the first part of the curves for Load histories A and B evidence the same behavior until step 45, where the joint in the left border of the upper arch opened. Consequently a horizontal displacement can be observed in the displacements curved. From load step 45 to 60, it can be appreciate that under those load histories the monitoring point tends to move more in Model 1 than in Model 2. The reason for that could be that the width of the opening that take place in the left side is bigger in Model 1 than in Model 2, since in the last one the displacements are reduced by the rods.

Again, for Load history C and D and expansion movement take place. Particularly, for load history C it can be observed a horizontal displacement followed by an increase in the slope of the curve which represent the separation of stones that take place in the upper part of the arch on the right side. The curves in those load histories are the same for model 1 and 2 because the rods are located in the lower part of the arch and do not able to avoid the opening of the joints in the upper part.

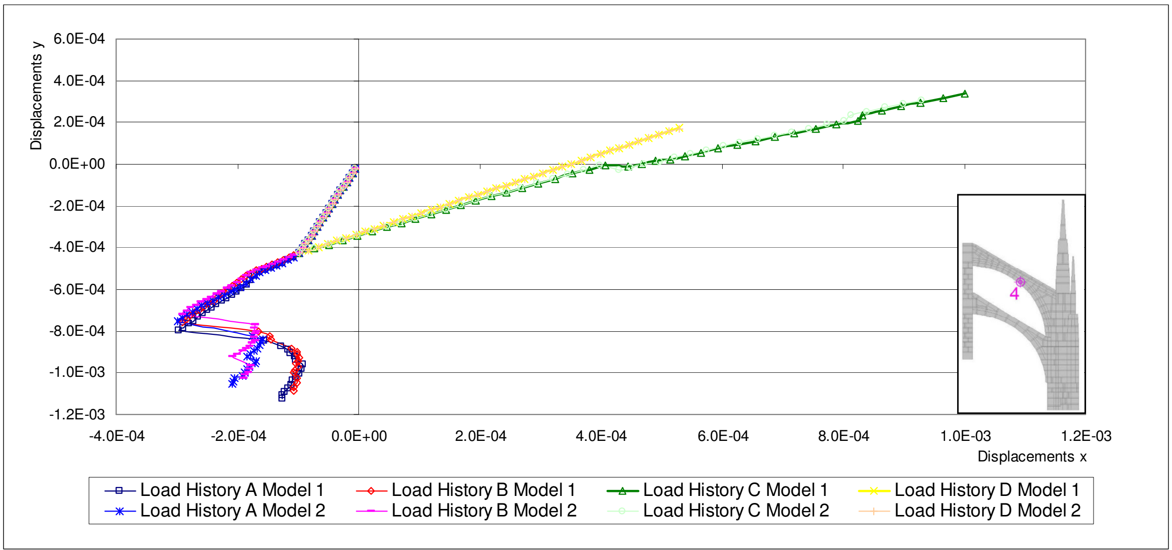

Figure 94 Monitoring point 4

Figure 95 shows the displacements of monitoring point 5 which is located in the upper part of the pinnacle. Once more the curves begin with a common line followed by an expansion movement for load histories C and D. In this case for Load histories C a concentrated horizontal displacement take place representing the opening of the joints in the superior part of the upper arch. After that the pinnacle continues its expansion with a higher slope. For load histories A and B the same behavior explained in monitoring 4 takes place.

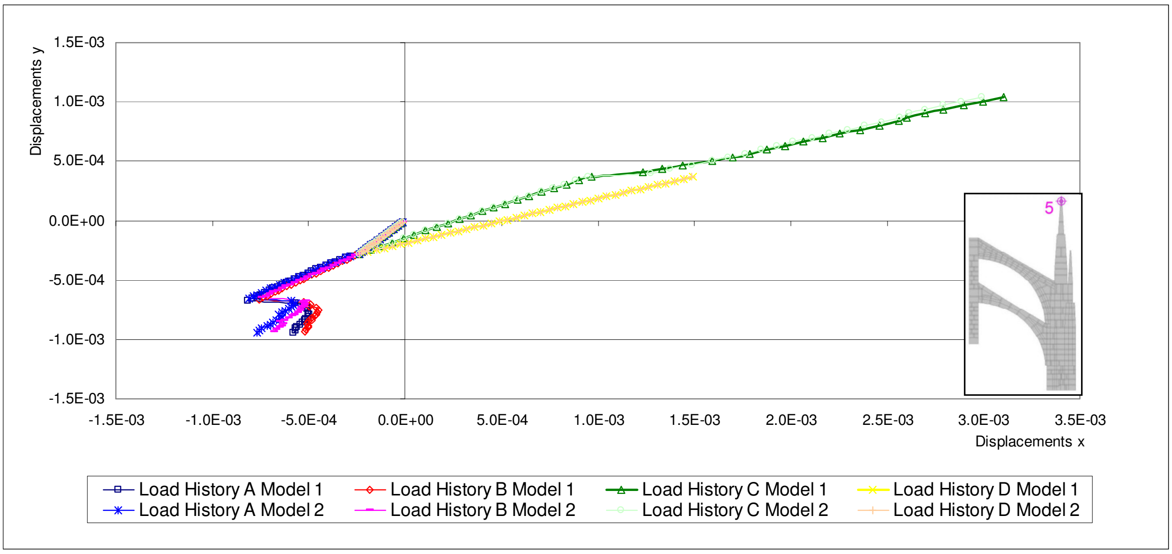

Figure 95 Monitoring point 5

11.4 Conclusions

It can be concluded that temperature have a big impact on the structure. However, it can be said that the intervention has a positive influence on the structure since it improves the secure of the joints against shear but do not change the structural behavior of the structure.

It can be observed that model 2 shows smaller values of deformation, in particular for load histories of cooling. The strengthen delay the process of opening of joints for the ones that star to open from the inferior part of the arches but do not influence much the behavior of the joints that begin to open from the superior part of the arches. From the displacements curves of monitoring point 2, 3 and 4 it can be notice that the lower arch has higher vertical movements than the upper one.

The upper arch is less influenced by the reinforcement than the lower arch. On the whole, the shapes of the deformed structures are similar in both models. However, the deformations of the arches in the reinforce model present more places with opened joins but with a smaller deeps; where as in the original system the opened of the joint take place in localize positions.

The repair avoids the increasing of stresses because the structure is allowed to deform. Though, it prevents the reinforced joints to open completely evading the increase of stresses due to the reduction of contact areas.

It is important to point out that cracks appear in the joints and not in the stones, what is reflected in the model as joint openings instead of cracks.

I would like to highlight that as mentioned before, the first joint on the lower arch on the left side is not reinforced. It seems to be that this decision was taken because of the special arrangement of the stones that are present in this position, which form a kind of step that could prevent the slip between stones caused by shear. This joint is particularly affected by temperature loads, as can be clearly notice on the models. Moreover, this joint is vital for the stability of the buttress.

Following the minimal intervention theory it is understandable that this joint was not reinforce. However, it is important to notice that the restoration was carried out in very respectful way and it is invisible from the outside. Moreover, the geometrical arrangement is not as reliable as the physical joint generated by the stainless bars and is not able to guaranty stability outside its plane. In addition, the place were this joint is located is not easily accessible. As a consequence, in the case of needing to reinforce this joint later, the cost of placing the scaffolding should be extremely high. Because of all those reasons, I believe that the joint under discussion should have been reinforced too, in order to guaranty the stability of the arch in the same way that in the other joints.

12. PROPOSAL OF FURTHER TASKS

Before mention any proposal of further tasks, it should be said that the current state of Saint Barbara church is good. In particular the stability of the structure has been improved trough the strengthening of the buttresses. From this point of view, the aim of future tasks should be to guaranty the conservation of the current state and if possible to improve it.

In historical constructions it is crucial to be aware that the time window of the structure is really long and that the current restoration is only a very short part. A maintenance plan to avoid future damages is vital to keep the current state of the church. In addition monitoring should be performed to evaluate the effectiveness of the restoration works. In this thesis some further interventions are also suggested, like the restoration of the façade and the interior of the church.

12.1 Monitoring

The monitoring plan that is proposed includes the study of three different phenomena.

The first phenomenon that is under study is the behavior of the crack that is located in the interior of the church at the level of the tribunes. The measurement of its width has been monitored for the last two years. Continuing with this monitoring for at least three more years is encouraged, because after following the movement of the crack for five years a conclusion about its behavior can be made.

It is expected to find out that the crack follow the same cycles of opening and closing during winter and summer correspondingly, not showing an increment in its width. In this case no intervention should be made, since fill in the crack will be particularly difficult due to the necessity of a material that should be sufficiently elastic to adapt to the cycles of opening and closing. If it is found out that the width of the crack is increasing, a deeper analysis should be made to explain the cause of the phenomena.

The second part of the structure that is suggested to be under monitoring is the reinforced buttress of the façade on the south side. In particularly, points were defined on this buttress after the restoration. The monitoring of the displacements of those points had been done during the last year to evaluate the effectiveness of the restoration and the response of the structure to the intervention. It is suggested to go on with these measurements. In addition, especial attention should be paid to the left joint of the lower arch, which has not been reinforced. Monitoring the horizontal displacement of this joint will allow assessing the need of strengthening this joint or not. Moreover the results obtained from the rest of the points will allow a complete understanding of the behavior of the structure after its restoration.

Due to the origin of the problem on the buttress, it is essential to consider that it is not possible to eliminate it. As a result, the structure will always be affected by cycles of temperatures and to stresses. After some cycles of temperature, damage accumulation is possible. Consequently the monitoring of the behavior of the buttresses is particularly important in Saint Barbara church.

The global change of temperature that is taking place nowadays in the word should no be neglected, since the values of maximum and minimum temperatures on the structure can be amplified. As a consequence the stresses introduced in the structure could increase and the structure will need to deform more leading to eventually failure of materials. The third phenomenon that is proposed to be under study is temperature variation on the surfaces of the flying arches. Monitoring of the temperature changes is encouraged to be done three times a year to evaluate if the temperature values remain in the same range or are increasing.

12.2 Maintenance plan

It is know that regular maintenance is the simplest way of ensuring the conservation of the building since it can reduce the necessity of repairs and replacements. Good maintenance anticipates problems and do not create new ones. Moreover, through regular inspections the development of defects can be found and remedied as quickly as they occur. [2]

The maintenance plan suggested to be followed can be divided in two groups according to the frequency in which they are expected to de done.

The first group makes reference to the maintenance routine which is propose to be done monthly. The works that are included in this group are cleaning gutters, downpipes, roof coverings and flashings to keep them in good working order; and controlling and removing the presence of birds and plants.

The second group refers to regular visual inspection. In this case it is expected that a visual survey will be done every year by a person with sufficient preparation in the area of conservation. This survey will include the inspections of the structural condition of the building and the condition of the interior and the exterior of the building. If a problem or decay is notice during the visual survey, a report should be written including the description of the problem, its seriousness and the procedure that should be follow to find out the causes of the problem or to solve it. Every five years the inspector should write a report about the state of the building. If necessary, the report should include programmed repair works.

12.3 Suggested interventions

The current state of the church is particularly good, however some interventions are suggested.

The façade of Saint Barbara church is the only part form the exterior that have not been restored in the current intervention. As explained before, there are some small cracks around the windows of the façade that should be filled in order to avoid its propagation. The cleaning of the façade is also proposed. Special attention should be paid to the floor of the balcony of the façade due to its decay condition, its intervention is necessary. The asphalts layers are broken and allow the rain water to penetrate to the interior of the church. It is suggested to remove all the old layers of asphalt using mechanical methods and where necessary chemical product to dissolve them. In order to protect the interior of the church it is recommended to cover the floor of the balcony with lead.

The intervention of the interior of the church is desirable. As said before, the causes of the humidity spots and layering of the materials have been treated. However to improve the aesthetical aspect of the interior and to highlight the design of the vaults, it is proposed to clean of the humidity spots in the vaults and in the wall. After cleaning a new plaster should be place.

13. CONCLUSION

From the studies that had been done it can be concluded that the church present a good state regarding its stability. Moreover the exterior of the church is also in a very good condition. The intervention of the façade in the near future is necessary in order to avoid future damages. Regarding the interior some minimal interventions are also suggested in order to preserve and highlight the outstanding design of the vaults.

It can also be concluded that the cause of damaged of the buttresses is temperature. Moreover, it should be said that even when the structure looks strong from the outside, its original design was not adequate since the section of the flying arches are different and consequently they present different thermal mass. Moreover, the upper arches have bigger section than the lower ones despite the fact that the lower arches take much more load than the upper ones.

From the results of the models, the effectiveness of the strengthening was proved. Moreover, it was notice that the stresses on the reinforcements are small and far from reaching the yield stress, consequently they would be able to resist eventual increasing of stresses.

The restoration show respect to the structure authenticity and the original structure behavior since it was reinforced but not changed. It is important to remark that the design of intervention consider the cause of damage. According to that, an increment of the rigidity of the arches was avoided since the stresses introduced in the arches by temperature increase with their rigidity.

The current restoration show respect to cultural heritage, as can be seen in the deeply research that has been done before the interventions works. Those works include active conservation task, such as cleaning, consolidations, replacements of stones; and preventing conservation, as improvement in the draining system and leading flashings. The restoration preserves the authenticity of the building including the artistic values as the macaroons.

During this work many of the tools available to investigate a historical structure were used: historical research, test and structural analysis. Those tools were combined to get an integrated conclusion.

I would like to highlight that for the city of Kutná Hora the church of Saint Barbara represent nowadays an economic resource as cultural and touristic attraction, therefore the need of preserving this building is not only a cultural requirement but also an economical and developmental demand.

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