SAHC

ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTION

Master's Thesis

Chandrashekhar Mahato

Reliability analysis of St. Barbara's Church in Otovice

University of Minho

Czech Republic | 2019

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CZECH TECHNICAL UNIVERSITY IN PRAGUE
Faculty of Civil Engineering
Thákurova 7, 166 29 Prague 6, Czech Republic

MASTER'S THESIS PROPOSAL

study programme: Civil Engineering

study branch: Advanced Masters in Structural Analysis of Monuments and Historical Constructions

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Student's name and surname: Chandrashekhar Mahato

Department: Department of Mechanics

Thesis supervisor: Pavel Kuklík

Thesis title: Reliability Analysis of St. Barbara´s Church in Otovice

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Framework content: The main objective of this thesis is to determine the stability of St. Barbara's church and elaborates upon the different methods adopted to evaluate the structural safety based on a probabilistic approach. A deterministic assessment of the structure is carried out and the results are assessed with reference to the present site condition. Depending upon the observed damages, a condition for failure is defined for the structure. Reliability analysis is then carried out taking into account the uncertainties in material parameters and to determine the reliability index, probability of failure and influence of different material parameters on the structural stability.

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.... Master's thesis supervisor

.... Head of department

Date of Master's thesis proposal take over: July 2019

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DECLARATION

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Supervisor(s):	Prof. Ing. Pavel Kuklik
Year:	2018-2019

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, Prague
Date:	05/07/2019
Signature:	___________________________

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Dedicated to my family,

friends and faculty members of SAHC.

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It was indeed a great pleasure and a moment of immense satisfaction for me to have completed this project.

Firstly, I would like to express my sense of gratitude and profound indebtedness towards my supervisor Prof. Pavel Kuklik, for his continuous support of my thesis, for his patience, motivation and immense knowledge. His zeal and enthusiasm have always been a source of inspiration for me.

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ABSTRACT

The monuments of Czech Republic constitute the most important part of the Czech cultural heritage. It symbolizes the significant historical development and provides valuable information about the behavior, cultural beliefs and economic condition of the society, from the most ancient time to the present day. The churches of Broumov, that portrays a unique Baroque architecture, is well-known not only for its distinct shapes and sizes, but also because of the religious, creative, artistic and historic values that it has carried over the years. The St. Barbara’s Church built in the year 1726 and located in the Otovice village of Broumov depicts similar architectural features and has been taken up as a case study for this project.

The main objective of this dissertation is to determine the stability of the structure through reliability analysis. Currently, most of the existing structures are verified using simplified deterministic approaches that are based on partial factor methods, usually applied in the design of new structures. Although the deterministic approach is widely accepted, yet more pragmatic information about the actual performance of the existing structure can be achieved through probabilistic method.

The report elaborates upon the various methods implemented to evaluate the safety of the structure based on probabilistic approach. In order to perform reliability analysis, the geometry of St. Barbara’s church was studied in detail, followed by a thorough damage assessment of the present condition. Based on the geometry, a deterministic finite element model was built and validated in GiD-Atena software. Further, the data obtained from damage assessment, formed the basis for the selection of failure criteria for the structure. The condition of soil at site was studied and the probability of failure was calculated by taking into account the effects of differential settlements. The dissertation also focuses on the uncertainties of material parameters and their influence on the structural stability and reliability. The reliability study has been performed using software packages SARA-FReET. Also, a comparison has been established to the existing codes and standards for the obtained reliability index. Subsequently, ways to better assess the current state of the church using probabilistic methods has been expressed.

Keywords: Cultural heritage, Reliability index, Probabilistic approach, Structural reliability, Uncertainty

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ABSTRAKT

Velkolepé historické stavby představují jednu s nejvýznamnější součástí kulturního dědictví České republiky. Symbolizují historický vývoj a poskytují cenné informace o chování, kulturním cítění a ekonomickém stavu společnosti, od nejstarších dob až po současnost. Broumovské kostely, které zachycují jedinečnou barokní architekturu, jsou známé nejen svou tvarovou rozmanitostí a velikostí, ale také náboženskými, tvůrčími, uměleckými a historickými hodnotami, které se vytvářely v průběhu let. Kostel sv. Barbory postavený v roce 1726 v obci Otovice v Broumově, který zobrazuje tyto architektonické prvky, jsem si zvolil a analyzoval ve svém projektu.

Hlavním cílem této práce je ověřit únosnost a stabilitu stavby s využitím spolehlivostní, stochastické, analýzy. V současné době je většina staveb posuzována pomocí zjednodušených deterministických přístupů, které jsou založeny na metodách dílčích součinitelů. Tento přístup je v současné době standardní při návrhu nových konstrukcí. Ačkoliv je tento deterministický přístup využíván, pragmatičtější informace o skutečném stavu stavby dosáhneme pomocí metod pravděpodobnostních.

Diplomová práce rozpracovává rozmanité metody založené na pravděpodobnostním přístupu, které byly využity ke zhodnocení bezpečnosti konstrukce. Aby bylo možné provést analýzu spolehlivosti, byla detailně prostudována geometrie kostela sv. Barbory, a následně popsán současný stavebně technický stav. S využitím těchto poznatků byl v softwaru GiD-Atena vytvořen a metodou konečných prvků validován deterministický model. Takto získané informace, zobrazující stavu poškození, se staly základem pro výběr kritérií, pro hodnocení porušení konstrukce. Na základě studia základových podmínek v dané lokalitě a byla počítána pravděpodobnost poruch zejména s přihlédnutím k účinkům nerovnoměrného sedání.

Diplomová práce se též zaměřuje na nejistoty materiálových parametrů a jejich vliv na stabilitu a spolehlivost konstrukce. Studie spolehlivosti byla provedena pomocí softwarových produktů SARA-FReET. Pro získání indexu spolehlivosti bylo také provedeno srovnání s existujícími kódy a standardy. Následně byly zmíněny postupy, jak lépe zhodnotit stav kostela s využitím pravděpodobnostních metod.

Klíčová slova: Kulturní dědictví, Index spolehlivosti, Pravděpodobnostní přístup, Strukturální spolehlivost, Nejistota

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सारांश

चेक गणराज्य के स्मारक चेक सांस्कृतिक विरासत का सबसे महत्वपूर्ण अंश हैं। ये स्मारक न केवल चेक गणराज्य के ऐतिहासिक विकास का प्रतीक है बल्कि अत्यंत प्राचीन काल से वर्तमान समय तक समाज के व्यवाहरिक तथा आर्थिक स्तिथी एवं सांस्कृतिक मान्यताओं विषय में बहुमूल्य जानकारी प्रदान करते है। ब्रूमोव के चर्च, जो एक अद्वितीय बारोक वास्तुकला का चित्रण करते हैं, न केवल अपनी विशिष्ट आकृतियों और आकारों के लिए, बल्कि धार्मिक, रचनात्मक, कलात्मक और ऐतिहासिक मूल्यों के कारण भी प्रसिद्ध है। सन 1726 में निर्मित बारबरा चर्च ब्रूमोव के ओटोविस गांव में स्थित है तथा इसमें इसी प्रकार के वास्तुशिल्प को दर्शाया गया है।इसी स्मारक का चयन वर्तमान परियोजना में अध्ययन हेतु किया गया है ।

इस शोध प्रबंध का मुख्य उद्देश्य विश्वसनीयता विश्लेषण के माध्यम से संरचना की स्थिरता का निर्धारण करना है। वर्तमान में, अधिकांश मौजूदा संरचनाएं सरलीकृत नियतात्मक दृष्टिकोण का उपयोग करके सत्यापित की जाती हैं । जो आंशिक कारक विधियों पर आधारित है तथा इन्हे आमतौर पर नई संरचनाओं के डिज़ाइन हेतु प्रयोग किया जाता है । यद्यपि नियतात्मक दृष्टिकोण को व्यापक रूप से स्वीकार किया जाता है, फिर भी मौजूदा संरचना के वास्तविक प्रदर्शन के बारे में अधिक व्यावहारिक जानकारी संभाव्यता विधि के माध्यम से प्राप्त की जा सकती है।

यह प्रतिवेदन संभाव्य दृष्टिकोण के आधार पर संरचना की सुरक्षा का मूल्यांकन करने के लिए कार्यवित विभिन विधियों का विस्तार से विवरण करती है।विश्वसनीयता विश्लेषण करने के लिए, सेंट बारबरा के चर्च की ज्यामिति का विस्तार से अध्ययन किया गया, इसके बाद वर्तमान स्थिति का गहन क्षति मूल्यांकन किया गया। ज्यामिति के आधार पर, एक नियत परिमित तत्व मॉडल का निर्माण किया गया तथा इसे GiD-Atena सॉफ़्टवेयर के माध्यम इस स्तापित किया गया । इसके अलावा, क्षति के आंकलन से प्राप्त आंकड़ों के आधार पर संरचना की विफलता के मानदंडों का भी विश्लेषण किया गया । संरचना स्थल पर मृदा / मिटटी की स्तिथि का अध्ययन किया गया तथा अंतर बस्तियों के प्रभावों के आंकलन से विफलता के कारको /सम्भावनाओ की गणना की गयी | अनिश्चितताओं तथा संरचनात्मक स्तिरथा एवं विशवसनीयता पर उनके प्रभावों की और भी ध्यान केंद्रित करता है । इसके अलावा,एक तुलनात्मक अध्यन के माध्यम से मौजूदा कोड एवं मनको के विश्वसनीयता सूचकांक प्राप्त किये गए ।

क्लीवोव नारा: सांस्कृतिक विरासत, विश्वसनीयता सूचकांक, संभाव्य दृष्टिकोण, संरचनात्मक विश्वसनीयता, अनिश्चितता

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1. INTRODUCTION

The architecture of Czech Republic is renowned in the world today, primarily for its rich history and cultural diversity. The Czech architecture showed outstanding examples in every century, ranging from the gothic castles, churches and bridges from the age of Emperor Charles IV to the baroque palaces of the Habsburg Monarchy and the neo-classical and art nouveau styles of the 19th century townhouses.

One of the major contributions to the Czech architecture is the Broumov group of churches, which is a unique complex of ten churches, built in baroque style of architecture. The churches were constructed in quite a short time period, between 1709 to 1743, on the territory of Broumov monastery estate by the famous architects Christoph Dientzenhofer and his son Kilian Ignaz. Although the churches differs from each other in size, shape and style, yet collectively they form an amazing architectonic complex. The different churches are as listed below:

- Church of St. George and Martin, Martínkovice
- Church of St. Mary Magdalene, Božanov
- Church of St. Margharet, Šonov
- Church of all saints, Heřmánkovice
- **Church of St. Barbara, Otovice**
- Church of St. Anna, Vižňov
- Church of St. Michael, Vernéřovice
- Church of St. Jacob The Bigger, Ruprechtice
- Church of the Feast of the Cross, Horní Adršpach
- Church of St. Adalbert, Broumov Monastery

Among all the churches of Broumov, the church of St. Barbara's provides the first strong and dynamic interior. Christoph Dientzenhofer's conservative scheme, which was based on simply adding spaces was developed by Kilian Ignaz, who added continuous curved walls, dramatic differences in heights and sophesticated segments of architectural details. However, over the advent of time, the structure have been isolated and neglected; and the lack of maintainence has adversely affected the health of the structure. In order to prevent the structure from further degradation, it is very important to perform structural evaluation and conserve the heritage structure.

The evaluation of structure is one of the most important factor considered in the conservation and preservation of the built cultural heritage. A precise structural evaluation can benefit the structure by providing accurate solution for intervention. For instance, a conservative evaluation may lead to an increase in the level of intervention of the structure and therefore result in the loss of authenticity and integrity. Moreover, it can also add unnecessary cost and compromise the workability of the conservation project.

This calls for a need for the application of reliability-based assessment to existing structures. The method allows to determine the global probability of failure by relying upon the deterministic technique's ability to calculate the structural stability for a prescribed set of parameters [1].

The following study illustrates an overall framework of the techniques of reliability assessment with special attention to the available commercial software (Atena, SARA, FReeT) and their requirements, advantages and limitations.

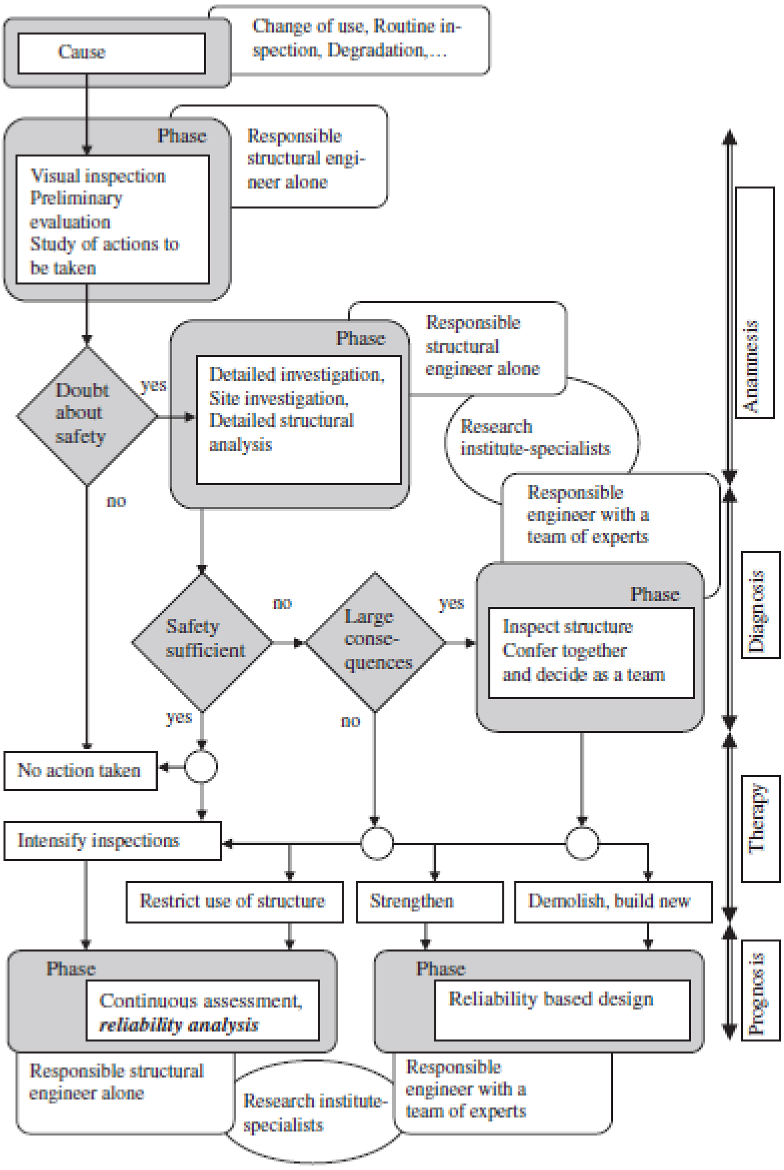


Figure 1.1 Process of preservation (Schueremans, 2001)

2. HISTORICAL SURVEY

Prior to the conservation and restoration, it is very important to have a glance into the background, geographical developments and major events experienced by the structure. Historical survey is a very important tool in the assessment of an existing structure as it can provide significant information regarding the materials, building typology and the change in use of the structure. The data obtained can be used to assess the present condition of the structure and identify the cause of damages.

2.1 Bohemia

Bohemia is the westernmost and the largest historical region of the present day Czech Republic. It was named after the Boii, a Celtic tribe that was constantly migrating into the Roman Empire. The ancient Germanic and Celtic languages were slowly replaced by the Slavic tribes that migrated from the east during the 6th century. It was only in the early 9th century that Christianity first appeared in the region, but however, it took several centuries to become a dominant religion. At that time, Bohemia was a part of Great Moravia, but after the death of the ruler, Svatopluk I, was under the dominion of the dynasty Premyslid, who remained rulers for the next centuries. The Dukes of Premyslid Vratislav II (1058) and Vladislav II (1158) proclaimed themselves as "King of Bohemia", but their descendants returned to the title of duke.

Bohemia had Charles IV as its first king, proclaimed in 1346, and also emperor of the Holy Roman Empire. The empire reached its economic and political heyday during the reign of Carlos IV. Prague was rebuilt and became known as the intellectual and cultural center of Central Europe. During his reign, Charles IV established Nové Město, founded Charles University in Prague and began with the construction of the bridge that crosses two banks of the Vltava river. In 1526, Bohemia became part of the Habsburg Monarchy, following the Battle of Mohacs.

At the beginning of the seventeenth century, a war began between various Protestant and Catholic states, generated by the imposition of the new Roman emperor, Ferdinand II, religious uniformity, and compulsion to Roman Catholicism. As soon as the Thirty Years War (1618-1648) began, a resulting revolt of Protestants in Bohemia. The result of this war was the destruction of entire regions and a high reduction in population numbers.

At the end of the 18th century, an attempt was made to recover the Czech language as a management rather than German language, but without any success. Just as in 1848 requests for autonomy from Bohemia were made in the Habsburg Empire. In 1871 there was a Czech attempt to create Austria-Hungary-Bohemia monarchy, though it resulted in failure.

After World War I, Bohemia was the basis for the newly formed country of Czechoslovakia. After World War II, the country was politically connected to the Soviet Union and led by the Communist Party.

Thus, becoming an administrative region of Czechoslovakia, since the country was no longer divided according to historical boundaries. With the end of Czechoslovakia, in 1993 Bohemia remained in the Czech Republic.

2.2 Baroque architecture

With the victory of the Catholics in the thirty years' war, a new architectural style, known as the 'Baroque' induced a new appearance to Czech architecture. The origin of the name Baroque is from the Portuguese word "barocco", which means an "irregular stone or pearl, but also refers to an idiom reflecting religious disturbances at that time.

The palaces and flamboyant churches built in this architecture style was designed to impress as a symbol of power of Catholicism. The new style was strongly supported by the upper class, and it provided to Czech architecture a new appearance after the War.

The buildings built during the early Baroque period of Baroque, were usually designed by Italian architects who were responsible for bringing this style from Italy - the cradle of baroque. The original Radical Baroque was created by Francesco Borromini and Guarino Guarini in Italy and built in Bohemia by Dientzenhofer and Jan Blazej Santini-Aichel.

Wallenstein Palace (Valdštejnský palác) was the first Baroque building in Central Europe designed by Giovanni Pieroni and Andrea Spezza, Italian architects.

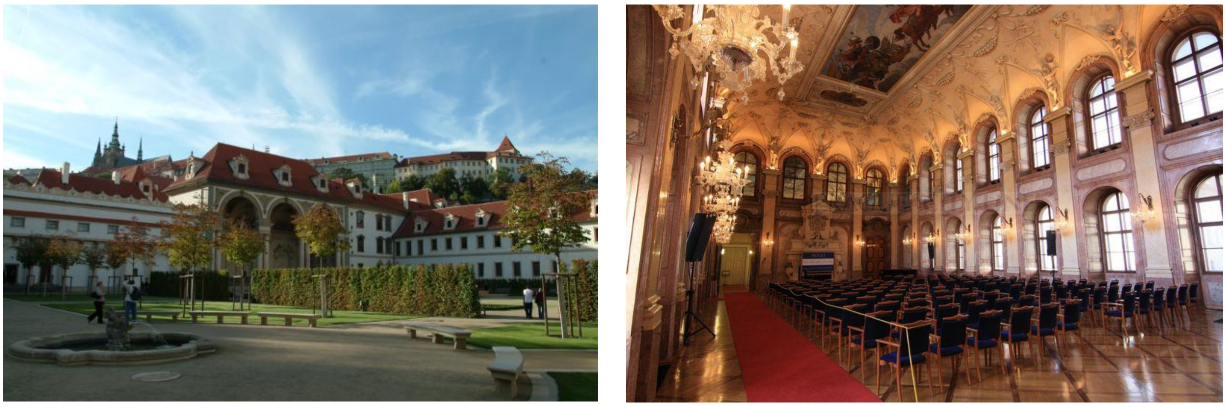


Figure 2.1: Wallenstein Palace

The Church of Our Lady Victorious, also referred to as the 'Shrine of the Infant Jesus', located in Lesser Quarter, was the first church built in Prague. This church was built in Renaissance style, but with damages suffered during the War was rebuilt in the early Baroque style. The construction was completed in the year 1613.

Figure 2.2: The Church of Our Lady Victorious, Prague

High Baroque, which was the next phase of this style, occurred between 1690 and the middle of the 18th century. This period had the two most significant architects, the father and son, Christoph Dientzenhofer and Kilian Ignaz. Their style was called "Radical Baroque", where they created a style to obtain movement through curved walls and intersecting oval spaces. Unlike the previous periods aimed at showing the reason in the architecture, Baroque tends to evoke distinct emotional states, most commonly dramatic.

In rejection of the symmetry of the Renaissance, the Baroque used several elements to express emotion and greatness as richness in decorations, grandeur, movement, and distortion of classical motifs. Baroque was used to make a strong impression of Church's domination and protection with some elements as light and shadows, the baroque architects implemented new forms to create a dramatic intensity in the buildings.

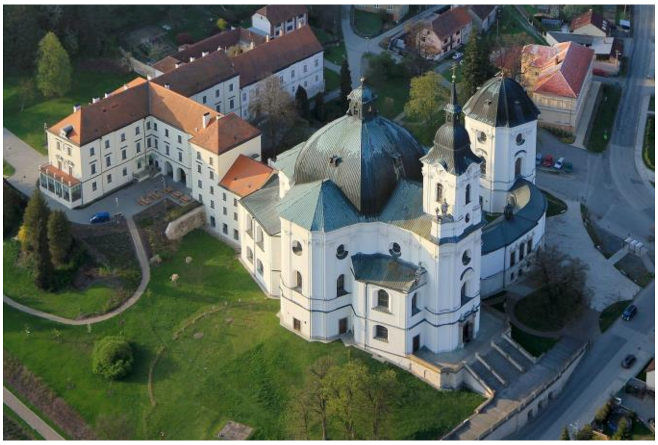


Figure 2.3: The front wall of Loreta in Prague by Christoph Dientzenhofer and Kilian Ignaz, 1666–1668 (left) and the iconic radical baroque space - Basilica Virgin Mary in Křtiny, Moravia, 1771 (right)

2.3 Broumov region

In the northeast of the Czech Republic, near to the border of Poland, lies the Broumov Region. As an integral part of Bohemia and despite the difficult events supported throughout its history, this region has managed to maintain some cultural values in terms of monuments. Its baroque monuments are of great architectural and religious importance, which characterizes the historical preciousness of Broumov Region.

Broumov was first officially mentioned in 1256. Further, the colonization of this region began in 1213 in Prague with the Benedictines of the Abbey of Břevnov, becoming a textile manufacturing center with sales market throughout Bohemia and Silesia. In 1348, it received privileges of administrative center of the abbey's manors by King Carlos IV.

Figure 2.4: Geographical location of Broumov region

According to the official website of Broumov city the "City layout is still preserved as the original Silesian type of town, with typical main streets connecting the whole built-up area and gates located on the opposite sides of the town. In 1357, the construction of city walls started and, with great difficulties and expenses, it finished in 1380".

The construction of the walls of the city began in 1357 and finished in 1380 with great difficulties and expenses. After a huge fire in the 14th century, the monastery and church dedicated to St. Adalbert (St. Vojtech) were built in the place where the old citadel was located. This strongly suggests that Broumov was one of the most important centers of Bohemia in the fourteenth century.

During the 16th century, the Thirty Years War took place (1618-1648). Characterized by the conflict between Catholics and Protestants, it ended with the victory of Catholics, this war left a great mark on the region. Despite the events taking place in Broumov the layout of the city is still preserved as the original city of Silesia, with the main typical streets linking the entire built area and gates located on opposite sides of the city.

After World War II, the country acquired a new policy, with imposed atheism, and hence the churches were neglected. Due to these facts, one can see the result of the lack of maintenance of these buildings in their current damaged state.

2.4 Broumov group of churches and Dientzenhofer influence

The landscape of Broumov's territory was influenced by prominent abbots in his construction works, such as Tomáš Sartorius from 1663 to 1700, and Otmar Zinke from 1700 to 1738. By the end of the seventeenth century, new churches were built in Broumov region by Martin and Giovanni Battista Allios.

The landscape of Broumov's territory was influenced by prominent abbots in his construction works, such as Tomáš Sartorius from 1663 to 1700, and Otmar Zinke from 1700 to 1738. By the end of the seventeenth century, new churches were built in Broumov region by Martin and Giovanni Battista Allios. Christoph and Kilian Ignaz Dientzenhofer (father and son) provided an extremely important contribution to the development of the Czech baroque style.

Christoph Dientzenhofer, known for using simple geometric forms, with rhythmic overlays like the propeller, came from the famous family of architects Dientzenhofer, being a respected Bavarian architect. Among his most famous works is the Monastery of Brevnov St. Nicholas Church in Prague, this was later completed by his son - Kilian, both buildings are located in Prague.

After 1709, Christoph Dientzenhofer was involved in the reconstruction of the monastery of Broumov. From 1719 to 1721, he was responsible for designing the Church of St. Michael in Vernéřovice, and from 1720 to 1723 to St James in Ruprechtice. His son, Kilian Ignaz Dientzenhofer completed the work of the St. Nicholas Church in Prague, being one of the main builders of the Bohemian baroque, one of his main works was the Kinsky Palace in Prague.

Kilian Ignaz Dientzenhofer, was also responsible for other churches in the region of Broumov: Church of All Saints in Heřmánkovice, in 1723; the extension of the presbytery of the church of St. John the Evangelist in Janovičky in 1725; Church of Santa Bárbara in Otovice, from 1725 to 1726; St. Anne's Church in Vižňov, from 1725 to 1727; Church of St. Margaret in Šonov, from 1726 to 1730, and the Church of St. Mary Magdalene in Božanov. The dynamics achieved with curvatures on the plants and various details are features recognized in Kilian's work (Chodějovská et al. 2015).

In the construction of the Churches of Broumov sandstone was widely used, quite abundant rock in the region due to its formation during the Mesozoic and Cretaceous period, when the sea entered the basin depositing calcareous clay (Facelli, 2014).

Figure 2.5: Churches of St Ann (left) and St Jacob (right)

The churches in the Broumov region are characterized by their similar construction, with a single nave low floor plan, with a simple decoration, due to the cheap and quick construction. According to Christian Norberg-Schulz, the probable economic problems of the churches construction provided less accurate details compared to other works by the same architects. In addition to these problems, all the churches in the regions were built on top of the hill or in the clear environment, which made it a visible landmark for the passengers of the main routes, dominating a village in its landscape. It was only in the nineteenth century that some churches were reformed and adapted to meet the new needs of the users.

3. ST BARBARA'S CHURCH

The Church of Santa Barbara is one of the cultural monuments of the Czech Republic. Located in Otovice, it is a Roman Catholic church and belongs to the Broumov group of Baroque churches.

Were founded some documents confirming the existence of an old church made of wood consecrated to St. George built in the same place as the Church of St. Barbara. According to the original sketch of Christoph Dientzenhofer, the construction of a new church began in 1725, but was modified by his son, Kilian Ignaz. The construction phase of the main part of the building took place in 1726. However, between 1748 and 1750, there were still works on interior design.

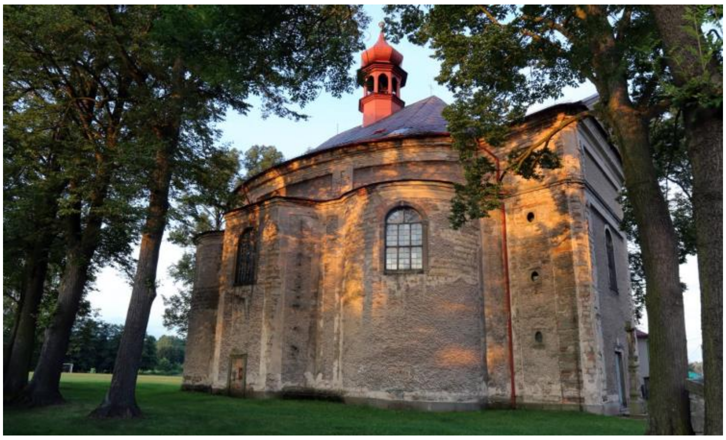


Figure 3.1: St Barbara's church, Otovice

The church is situated near the crossroads for Martínkovice approximately in the middle of the village, which follows the river Stenava and the road to Silesia. the church is visually connected to the nearby church tower in Matinkovice, its facade was placed near the main road.The original place and orientation of the older church was changed. The original church was built in a large open place, where the site subsequently exists.

3.1 Architectural Description

The exterior of the church has the same arrangement of the floor plan in an extended oval with eight additional spaces. The sacristy and the oratory are connected in the longitudinal axis, and the transverse axis is closed with five-octagonal pictorial spaces. the spaces on the diagonal axis are closed with a semicircle. the lateral spaces have differentiated heights and leave the walls of the enclosure decorating the simple side walls that are conceived as a uniform facade around the whole church. the input shaft is accented with two-story avant-garde with electrified floor, enclosed with a triangular beam.

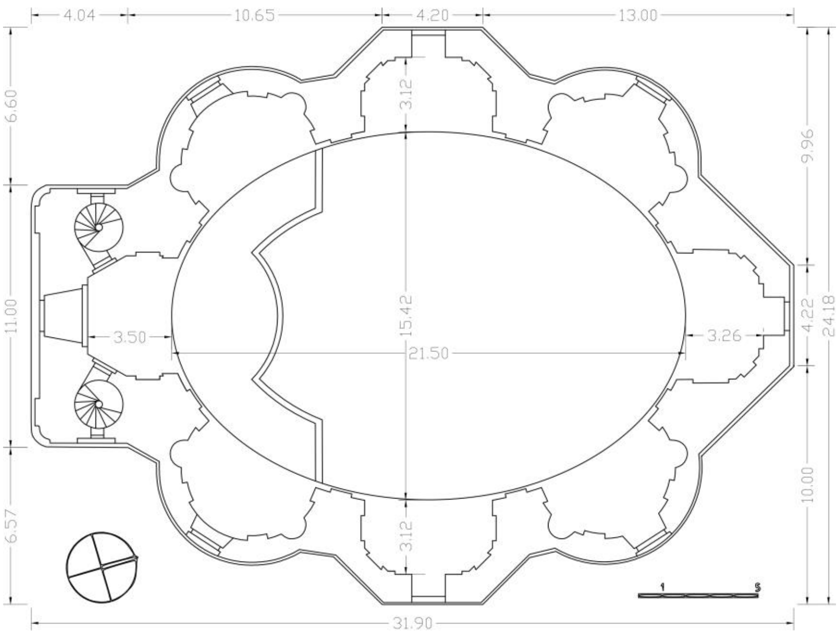
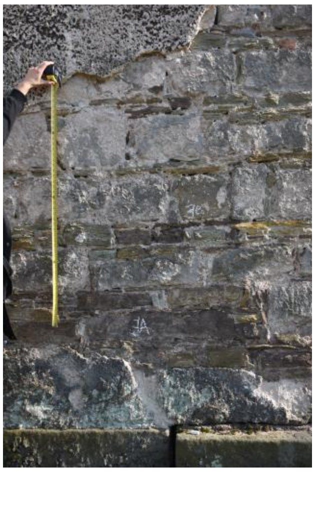


Figure 3.2: Floor Plan of St Barbara’s church (All dimensions in meters)

The interior of the church was built according to a well-established scheme already used in Vernérovice and Ruprechtice. as there was a straight wall of niches, in this case the building was moved to a more advanced type of continuous curved walls used by Cristoph for the construction of the chapel of St. Mary Magdalene in Skalka u Mnísku and in plans for a church in Rozmitál pod Tremesínem, which was not built.

3.2 Materials

The region of Broumov is rich in sandstones. The stones used for the construction of the St Barbara’s church is primarily built with claystone and sandtones that were obtained from the local quarries. The mortar of the structure is composed of lime. The roofs and the ceilings are made entirely with timber covered with slate/ shingles.



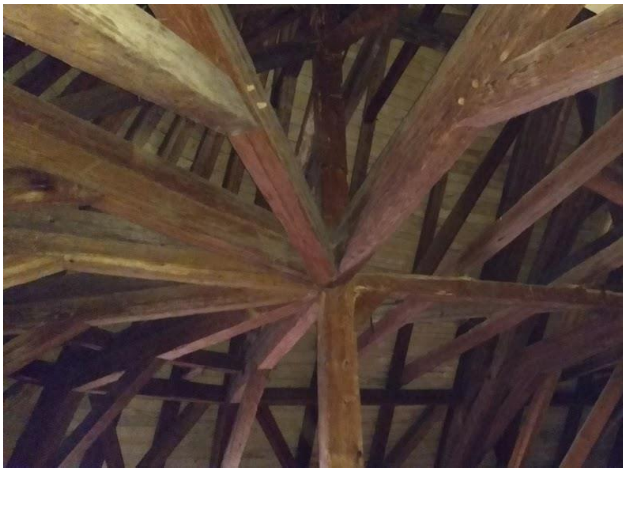


Figure 3.3: Building materials of St Barbara's church

3.3 Geological condition

The information on the geological condition of the church were obtained through borehole testing. The tests were conducted in two spots upto a depth of 12 meters below the surface. The location of boreholes has been shown in figure 3.3. The information obtained from the tests were analysed by the Department of Geotechnics (CTU, Prague) which provided the basis for calculating the soil stiffness and understanding the possibility of soil settlements. The underground water level was detected at a depth of 3.5 m below the surface on both boreholes.

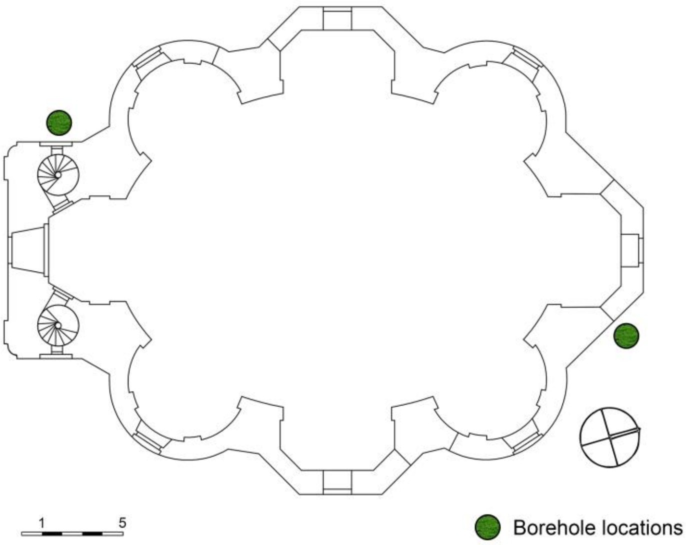


Figure 3.4: Location of boreholes for testing

The geology of Broumov is composed of bedrock that is formed only by rocks from Paleozoic era, which are mostly presented by Permian sediments such as Broumov siltsones, sandstones and claystones. These rocks were found relatively close to the surface and directly under the foundations of the church (Appendix A & B). The bedrock is mostly formed from more or less weathered silty claystones. The color of the stones changes from redbrown to gray claystones towards the bottom. Each layer has the different state of compacting and degradation due to the weathering action. The layer at the surface is represent by sandy soil with small fragments of claystones. The geological situation is in the following picture (figure 3.4).

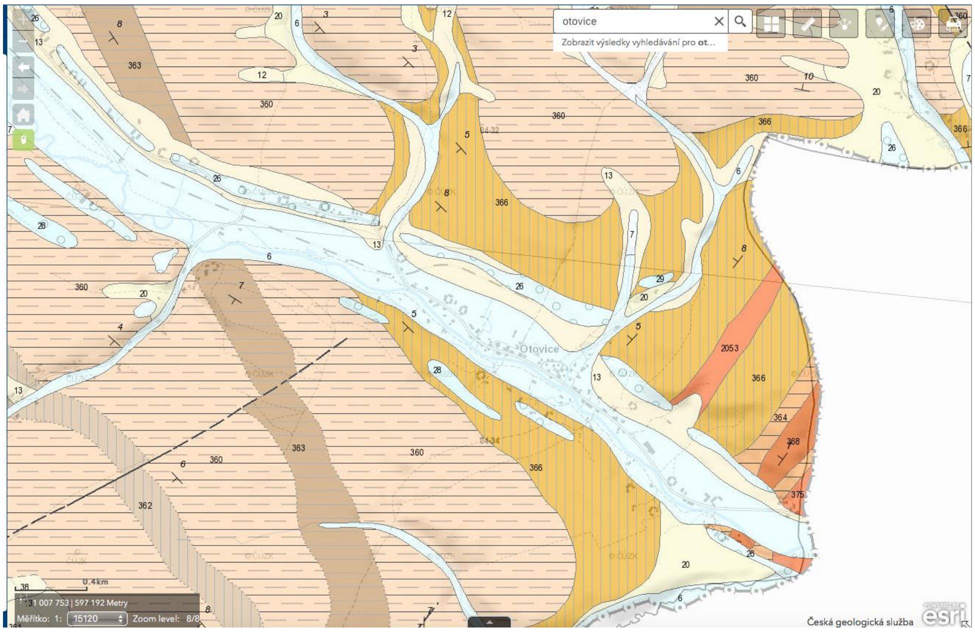


Figure 3.5: Geological map of Otovice

According to the inference provided by the Geotechnics Department, the stones that are closest to the site are gray to greenish gray silty claystones, in some places bituminous limestones, siltstones and finegrained sandstones (represented by the number 366 in figure 3.4). The other constituents of the soil are siltstones, clay siltstones, silty finegrained sandstones (360), andesitic tuffs and aglomerates (368), limestones, marlites, siltstones, andesitic tuffs (2053), aleuropelites, arcose and arocose sandstones positions, vulcanodetritic breccias, sandstones and tuffs (364), rhyolithic tuffs and rhyolites (375).

3.4 Climatic condition

The region of Broumov experiences cold and temperate climate. There is significant rainfall throughout the year in Broumov. Even the driest month receives ample amount of rainfall. This location is classified as Dfb by Köppen and Geiger. The average annual temperature is 6.9 °C in Broumov. Precipitation here averages 640 mm. At an average temperature of 16.2 °C, July is the hottest month of the year. In January, the average temperature is -3.9 °C. It is the lowest average temperature of the whole year. Between the driest and wettest months, the difference in precipitation is 63 mm. The average temperatures vary during the year by 20.1 °C.

4. DAMAGE ASSESSMENT

A damage survey was performed for the church of St Barbara based on visual inspection. The primary objective of the investigation was to find evidence of inferior or cracked walls, sloping floors, damaged structural elements, gaps between walls and ceilings, etc. The damages observed has been listed as follows:

4.1 Cracks

A crack is formed when the stresses acting on the structure overcomes the material strength. Also, the part between the crack could show some deformation.

Figure 4.1 Cracks in the internal (a) and external (b) (c) walls of the church

In the case of St Barbara, it is very important to distinguish between an active and a dormant crack. While dormant cracks remain relatively unchanged in size, active cracks are related to a non-stabilized phenomenon that can increase, causing damage or failure of the element/structure. Most of the cracks in the church apper to be dormant. However, it should be noted that the appearance of cracks, is not necessarily an indication of risk of failure in a structure. This is because cracks may relieve stresses that are not essential for equilibrium.

4.2 Discoloration and deposition

According to HeritageCare, discoloration corresponds to a change of the colour of the material surface in one to three of the colour parameters, hue, value and chroma. It may affect the surface or be present in the depth of the material.

Deposits correspond to an accumulation of materials on the surface, such as salts and pollution. This accumulation could have a homogeneous or non-homogeneous and a regular or irregular. It could have a weakly or strongly bonded to the substrate. And when it detached from the substrate or are removed can include some material of the substrate.

Figure 4.2: Discolouration and deposition

4.3 Degradation of soil

The structure shows signs of settlement in the exterior of the church. Some of the plinth stone close to the rainwater gutters has been detached or displaced from the wall.

Figure 4.3: Degradation of soil and settlement

4.4 Moisture

The excess of moisture can detoriorate the plaster and renders of a building. The presence of moisture in the surfaces allows the development of the organisms.

Figure 4.4: Effects of moisture and rising dampness in the church

4.5 Detachment

The detachment process usually starts from the surface of the material, and it could only affect the surface of the material or it could occur in depth. It corresponds to a physical separation that can correspond to a single or aggregates of grains, or a superficial layer with some thickness.

This damage in the church could have been caused by the combination of the action of rain, wind and water containing solid particles, volume variations with consequent internal pressures and chemical reactions.

Figure 4.5: Detachment and loss of material

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5. NUMERICAL MODEL

5.1 Geometry

The geometry of the structure was constructed after detail measurement at site with the help of measuring tapes and laser meter.The ground floor plan was referred to the plans obtained from the book Kilian Ignac Dientzenhofer and the artists of his circle. The 3D model for numerical analysis was built in AutoCAD. Several assumptions were made to simplify the geometry of the model. Non structural architectural elements such as cornices and decorations have not been considered for the model. The thickness of the domes above the windows in the transepts were assumed to be 35 cm thick. The semi-circular projections/ apses along the transept has been removed for further simplification of the geometry. Also, small niches and recesses in the walls were removed to prevent issues in mesh.

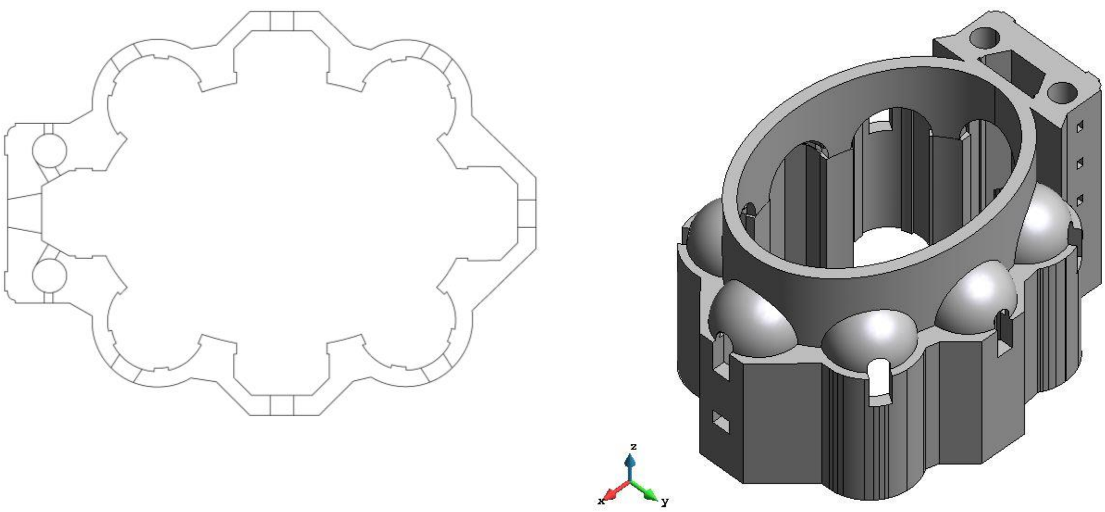

Figure 5.1: Simplified geometry (left) and numerical model (right)

The finite element mesh for the 3D model was generated in GiD, which is a pre and post processor for numerical simulation in Atena science and engineering, developed by Cervenka consultancy. The problem type was defined as Atena-Static with tetrahedral elements type.

5.2 Material Properties

For existing structures, the identification of the material properties is one of the most crucial and challenging part. Contrary to the modern structures, which are code based, structures in the past were built with uncontrolled and not manufactured materials. Thus, the properties of the material are unknown, resulting in the necessity of tests to define the material parameters. Today, a wide range of destructive, semi destructive and non-destructive techniques are available with each technique specific to a certain characteristic of the material. For example, in order to obtain the compressive strength of a masonry, destructive uniaxial compressive test can be considered as the most appropriate approach. Although destructive tests are most certain, yet it has its own limitations when it has to be performed on heritage structures and so, non-destructive tests are generally preferred over it. Non destructive tests usually provide qualitative information which can be utilized to obtain a range of possible properties. A combination of various tests can however provide the most suitable and realistic values. Furthermore, sometimes when none of the above tests is possible either due to large costs or inaccessibility to the structure, the material properties are estimated based on literature and engineering judgement (Vicente et al., 2016).

The material property and in-situ testings of St Barbara church were out of the scope of the present work, therefore the material data are based on the study conducted on a similar church (St. Ann church) at Broumov region (Gajjar, 2018). The mechanical parameters, considered for modeling has been listed in Table 1.

Table 1: Material properties for the numerical model

Material Properties	**Values**	**Units**
Youngs Modulus	2.0	GPa
Poisson's Ratio	0.2	-
Mass density	2.0	T/m^3
Crack Orientation	Fixed	-
Tensile Curve	Exponential	-
Tensile Strength	0.2	MPa
Tensile Fracture Energy	70.0	N/m
Compressive curve	Parabolic	-
Compressive strength	2.9	MPa
Compressive Fracture Energy	9100.0	N/m

Parameters such as peak compressive strength and Young's modulus has been obtained from the 2D numerical analysis of the wall section of St. Ann church. The tensile strength has been taken as 10% of the compressive strength and the compressive fracture energy has been computed from a ductility index (d) of 1.6 mm was used following Model code 90. Besides, a high value of tensile fracture energy was chosen in order to ease convergence in the numerical model.

5.3 Material constitutive model

The material constitutive model was defined in Atena-GiD interface with static problem type. However, the analysis is carried out in Atena Science, which is a software exclusive to mainly reinforced concrete structure. Hence, utter caution is required when applied to masonry. The constitutive model selected for the church was "Non linear cementitious 2" (Cervenka & Jendele, 2018). This fracture-plastic model basically combines the constitutive models for tensile (fracturing) and compressive (plastic) behavior. While the fracture model is based on the orthotropic smeared crack formulation and crack band model, the plasticity model is based on Menétrey-Willam failure surface. The model can be effectively used to replicate conditions like concrete cracking, crushing under confinement and closure of crack. [Atena Theory]. One of the peculiar characteristics of this model is that it is not confined to any particular shape of hardening/softening laws and can handle physical changes like for instance crack closure. Also, in this model the tensile behaviour is determined by an exponential opening law (figure 5.2), where it is necessary to define the fracture energy in tension.

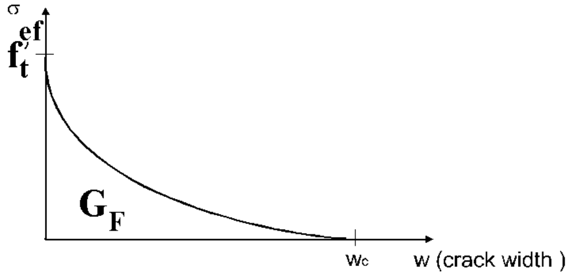

Figure 5.2: Exponential crack opening law

The function of exponential crack opening is expressed as follows (Hordijk, 1991):

$$\frac{\sigma}{f_t^{\prime ef}} = \left\{1+\left(c_1\frac{w}{w_c}\right)^3\right\}\exp\left(-c_2\frac{w}{w_c}\right)-\frac{w}{w_c}\left(1+c_1^3\right)\exp\left(-c_2\right),$$

$$w_c = 5.14\frac{G_f}{f_t^{\prime ef}}$$

where w is the crack opening and w_c is the crack opening at complete release of stress. σ is the normal stress in the crack and c_1 and c_2 are constants withthe valuess of 3.0 and 6.93 respectively. G_f is the fracture energy required to create a unit area of stress free crack and $f_t^{\prime ef}$ is the effective tensile strength acquired from a failure function.

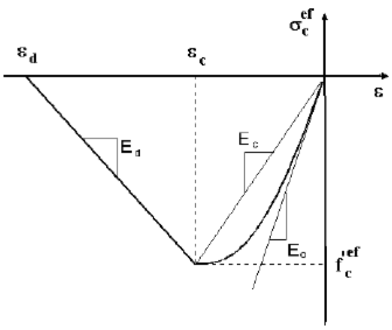

Figure 5.3: Compressive stress strain diagram

For compression, the strain at the peak stress has to be determined along with the critical displacement w_d. The softening law (post peak stress) in compression is linearly descending and is defined by two models, one is based on dissipated energy and the other is based on local strain softening.

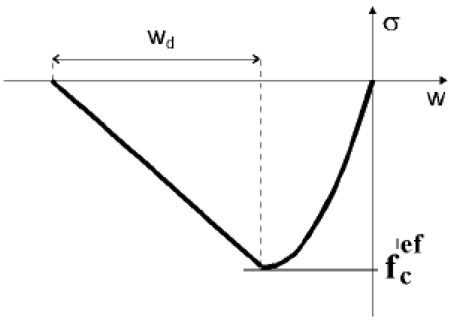

Figure 5.4: Softening displcement law in compression

5.4 Load

In order to understand the stresses that is being currently experienced by the structure, the following combinations of load were used:

5.4.1 Self weight of the masonry

The density of 20 kN/m^3 was taken for the masonry walls. Considering the average thickness of the walls as 1.4 m and height of the structure as 15.5 m, the linear load was computed equal to 434 kN/m.

5.4.2 Self weight of timber roof

The linear load distributed on the walls due to the timber roof is taken as 25 kN/m. This value was estimated from the past study conducted by Giulia Facelli on the timber roof of St Jacob church (Scacco, 2018). The total load includes the weight of the truss of the main structure, the roof cover and the beams supporting the platform and the ceiling. The roof cover and the ceiling contributed to a load of 0.93 kN/m^2 and 0.5 kN/m^2 respectively.

5.4.3 Live Load

As recommended in EN-1991-1-1, roofs that are not accessible except for normal maintenance and repair should be imposed with a load of 0.4 kN/m^2. However, according to table A1.1 of EN-1990, along with snow, the combination factor is taken as 0.

5.4.4 Snow Load

The characteristic value of snow load was taken to be 2.25 kN/m^2 as per the recommendation of the National Annexes (Czech Republic) for the region of Broumov.

5.5 Boundary conditions

For determining the boundary conditions for the deterministic analysis, the obtained information of the geology and soil characteristics were analysed using an approach similar to (Gajjar, 2018) and (Scacco, 2018) using Geo 5 software. A stiffness of 26.36 MPa was obtained for the soil. This was modeled into the numerical model as springs for support.

5.6 Deterministic Analysis

The quality of the finite element mesh plays a very important role in determining the results of the analysis. A large size mesh can prevent bending and make the structure more stiff. Therefore, it is very important to select a mesh of optimum size. For the deterministic analysis, 0.20 mesh size was selected, ensuring the generation of at least four tetrahedral elements along every thickness of the structure as recommended by [ATENA-Engineering_Example_Manual].

Since the boundary conditiions were set to allow settlement in the soil, a maximum displacement of 18 mm was obtained for the structure in the vertical direction.

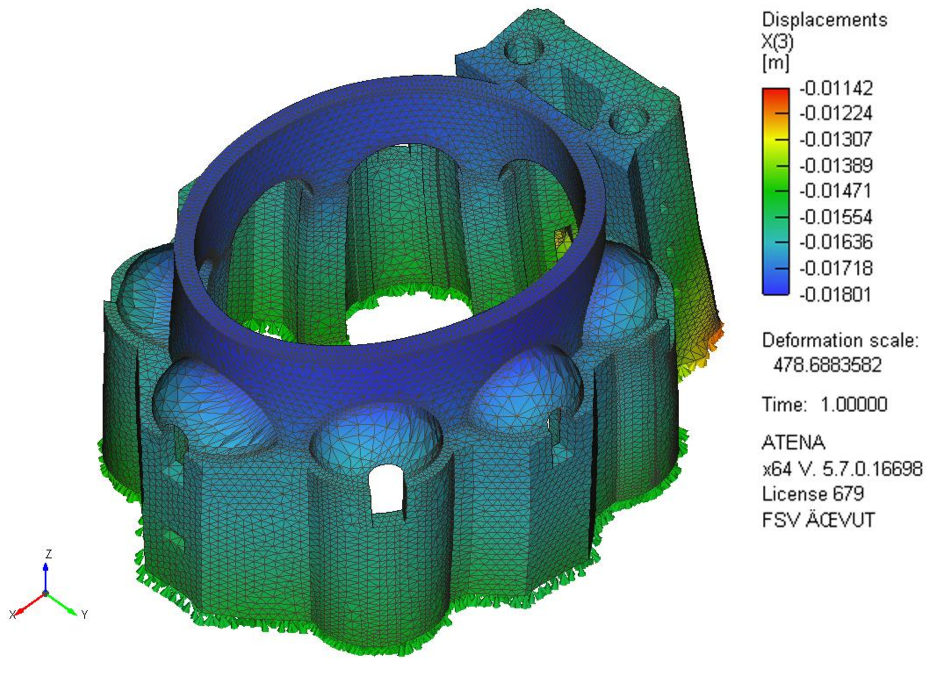

Figure 5.5: Displacement of the structure due to uniform settlement

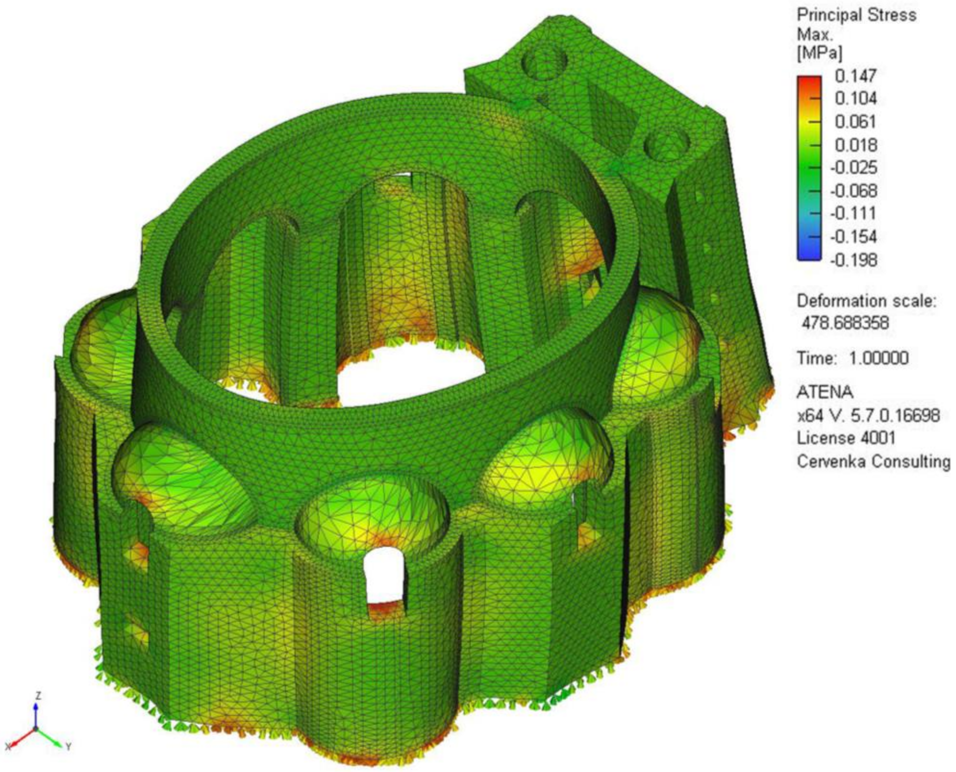

Figure 5.6: Maximum principal stress in the structure due to uniform settlement

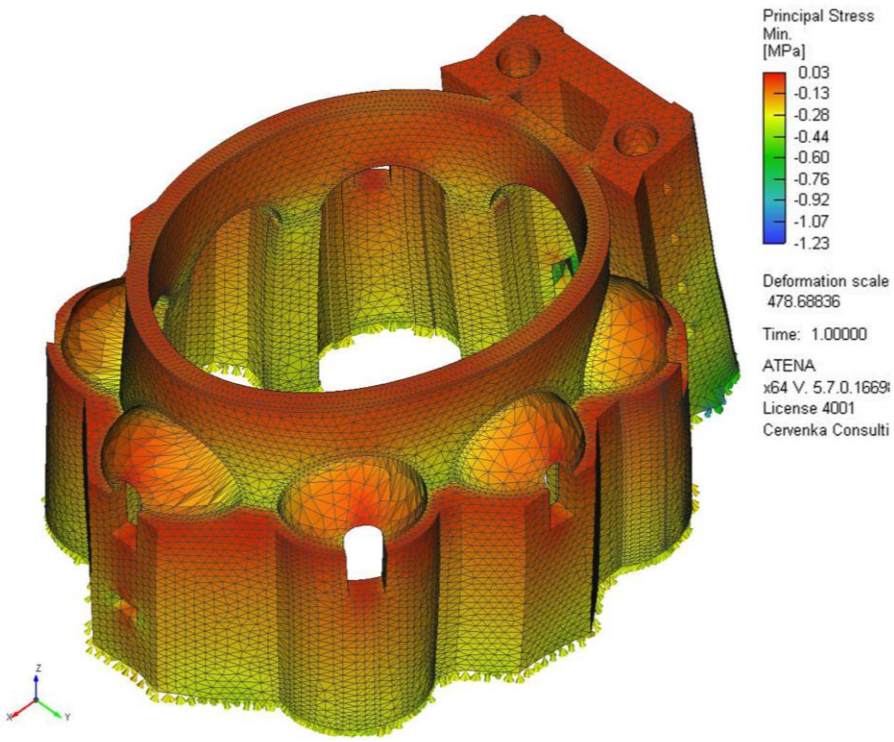

Figure 5.7: Minimum principal stress in the structure due to uniform settlement

6. RELIABILITY ANALYSIS

6.1 Probabilistic Assessment

Probabilistic based structural assessment plays a very important role in existing buildings as the various factors affecting risk are not precisely known. The deterministic method, which is the most widely used method of risk assessment tries to counter the uncertainty and variability by incorporating fixed safety factor. However, this fails to provide a complete information regarding the range of possible risks as it is difficult to decide the safety factors depending upon the structure.

The probabilistic approach basically takes into account the randomness in selected model parameters and the probabilistic distribution function (PDF) for the uncertainties of the solution. Then, based on the relationship between the applied load and the structural resistance, the reliability index is obtained which is an assessment of the safety and failure probability of the structure. It provides a more accurate mean to assess the performance of the structure (Matos, 2010).

Some studies conducted in the past proposes probabilistic based safety assessment considering both loading and resistance PDF as time variant quantities.

A probability density function (PDF) for continuous random variables can be defined as the first derivative of cumulative distribution function (CDF). It is the summation of all probability functions corresponding to the values less than or equal to the considered variable, defined as ‘x’ in the following equation (Milczarek, 2017):

$$F_x\,(x) = P(X \leq x)$$

The randomness in the numerical model has been considered as the uncertainties due to load. In order to measure the linear dependence between two input parameters, a correlation matrix comprising of the correlation coefficients of different parameters is used. The matrix is computed using the Pearson coefficient, which is calculated by dividing the covariance of two parameters with the product of standard deviation. This correlation value ranges from -1 to 1 and the closer it is to the value of -1 or +1, the stronger is the correlation between the two parameters (Matos, 2010).

When assigning the mean and standard deviation of the parameters in the PDF, statistical uncertainty is introduced into the probabilistic model. These uncertainties are later reduced when more information about the PDF of the input parameters is obtained. This upgradation in the probabilistic model is performed using the Bayesian inference.

6.2 Failure Criterion

Several models with various boundary conditions and monitoring points were developed to understand the different mechanisms of failure for the church. One of the initial models was based on calculating the maximum resistance of the structure by applying an incremental load. To incorporate this into the model, a uniformly distributed load was applied to the top of the church and divided into 10 steps to capture the linear and non-linear behavior of the capture. Monitors were placed to measure the displacements at the top of the tower and reactions at the base of the structure, which would be used to determine the maximum resistance.

However, due to the limitations of the software that was used for simulation (GiD-Atena v5), the load-displacement curve could not be plotted when springs were attached to the supports. This is because the program does not consider springs as a type of support, and is incapable of monitoring the summation of reactions at the springs.

To counter this problem, failure was defined for the structure in terms of crack width caused due to differential settlements. As mentioned earlier, the existing condition of the church shows that the foundation stones near some of the rainwater gutters have moved or detached from the structure. One of the probable causes of the damage could be due to the stagnation of water caused by non-functional gutters. Presence of water in the footing can cause an adverse effect on the structure.

The foundation of the church is mostly built from natural stone. Similar to any other building material, building stone undergoes weathering processes and looses its original qualitative properties, such as uniaxial compressive strength and durability. The underlying rocks can also change its properties such as cohesion, due to the weathering action over the period of time. In most cases, the stone materials of footing masonry and underlying rock are often similar. This is because the ancient builders generally used stone material from the nearby surrounding.

The ability of water in liquid state to dissolve chemical compounds is the most important factor controlling the durability of the natural stones. Compared to all natural stones, the clastic sedimentary stones are the most sensitive to weathering agent. This is basically due to the arrangement of its molecular structure, especially the abundant presence of interconnected pores.

The presence of water may also cause dissolution and leaching of the binders and thus adversely affect the cohesion. In the case of building constructions, which are made up from sedimentary stones, it is therefore necessary to eliminate the water income into the footing masonry and into the nearest underlying rock.

To consider the effect of differential settlement in the numerical model, vertical displacements were added in areas showing signs of damage or detachment in the foundation stones that were identified through visual inspection. The loads were then applied to the structure in two different intervals. In the first interval, the loads comprised of the self-weight and the dead load were applied to the structure, along with a spring stiffness of 26.36 MPa. This caused a uniform settlement of 18 mm to the structure as seen in figure 5.5.

In the second interval, a vertical displacement of 100 mm was applied progressively at the base of the foundation to simulate the effects of differential settlement. The mentioned displacement was applied in 10 load steps (figure 6.1) such that each step corresponds to 10 mm of displacement and the crack pattern was observed.

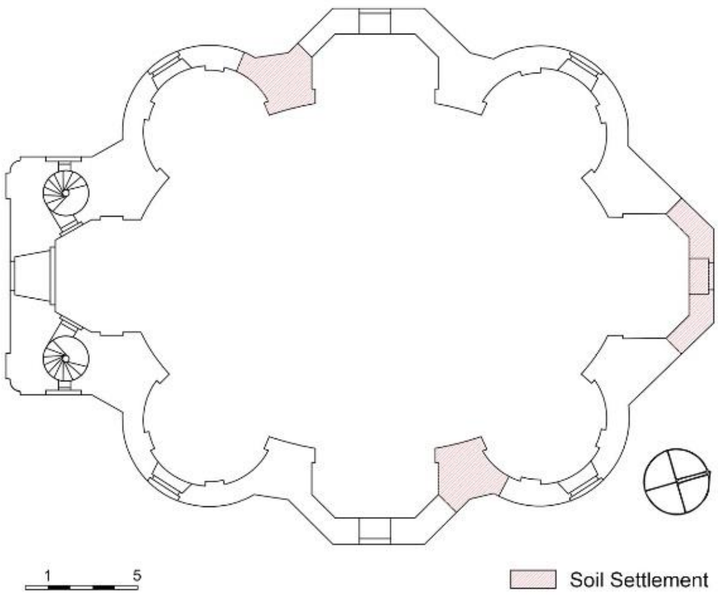

Figure 6.1 Location of application of soil settlement

The development of cracks at different load steps can be seen in the figure 6.2.

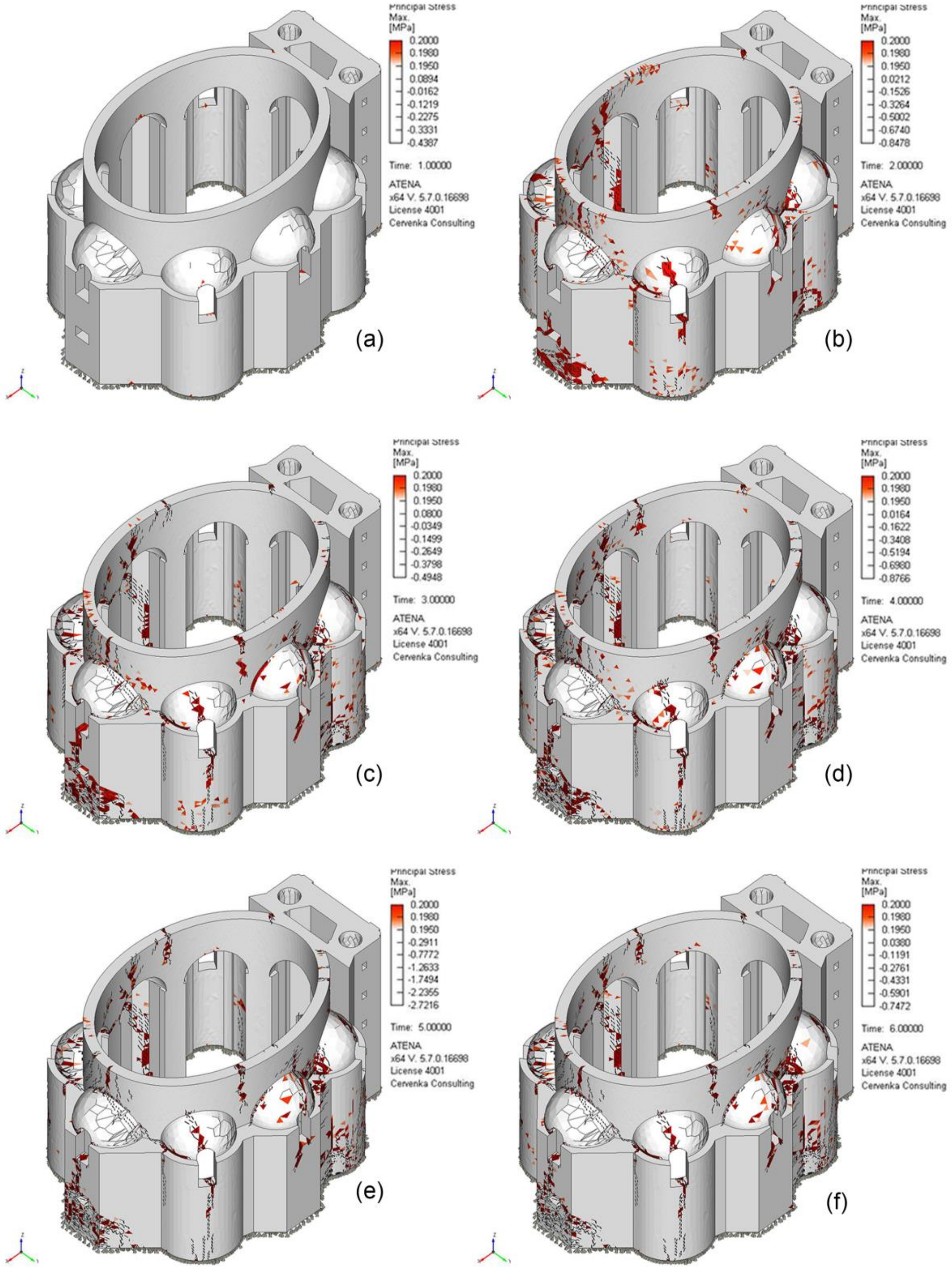

Figure 6.2: Maximum principal stresses and generation of crack at different load steps (a) step 1 (Uniform settlement), (b) step 2 (20 mm displacement), (c) step 4 (40 mm displacement), (d) step 6 (60 mm displacement), (b) step 8 (80 mm displacement)

It is important to note here that it is not necessary that all the cracks shown above contribute to failure of the structure. Therefore, the different types of cracks were correlated to their respective width as shown in figure 6.3.

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

Figure 6.3: Correlation between the wodth of the cracks and their level of danger (Scacco, 2018)

Cracks of width larger than 15 mm corresponding to 'severe' degree of damage and expected to disrupt the functioning of the structure was filtered from the smaller cracks (figure 6.4 b). It was observed that at load step 80 which accords with 80 mm displacement showed a complete arch-like crack formation of thickness 15 mm at the rear wall of the church (figure 6.4). It is important to note here that, although this load can completely detach the section of the wall under the window, the failure is very local, therefore it is possible that the superstructure would still prevent collapse. However, it is evident that the wall completely detaches itself from the superstructure and so, a differential settlement of 80 mm was taken as the failure criterion for the church.

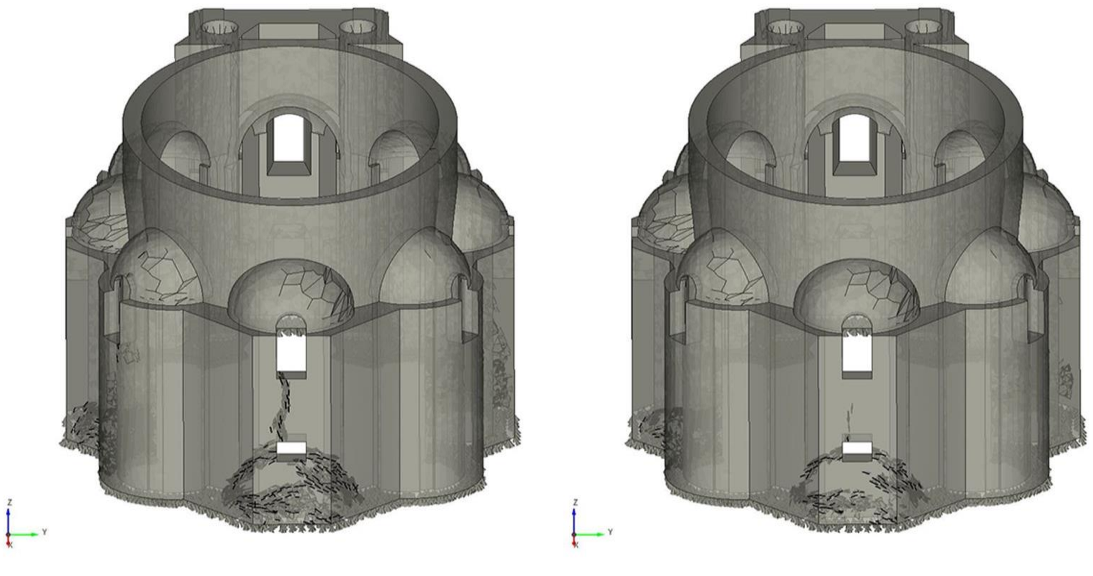

Figure 6.4: Filtered crack width showing (a) 5mm (Moderate), and (b) 15 mm (Severe) degree of damage

Once the criterion for failure was defined, it was important to generate a probability distribution function (PDF) for the ultimate limit state of the structure. As mentioned earlier, the software could not provide the total reactions for spring and so, the PDF generated for the reliability analysis was based on the stresses. The maximum tensile strength of the structure is 0.2 MPa, beyond which the finite element would start to show cracks in the numerical model. The element nodes in the finite element mesh that reach the peak tensile strength exactly at 80 mm of displacement, were identified and monitored for the maximum principal stresses. Basically, these nodes acted as a flags or checkpoints for the analysis to mark the point when the structure reaches the considered failure criterion.

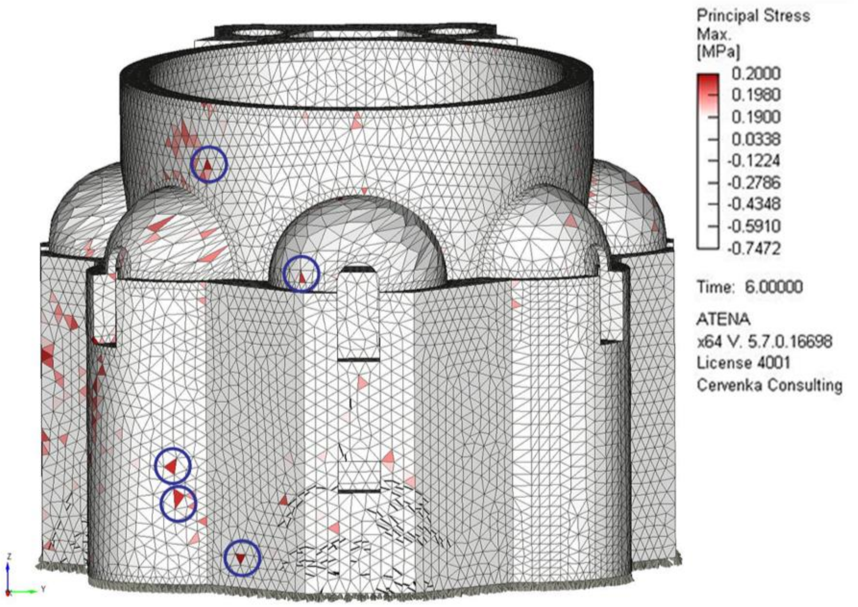

Figure 6.5: Selection of monitors for the church (marked with a circle in blue)

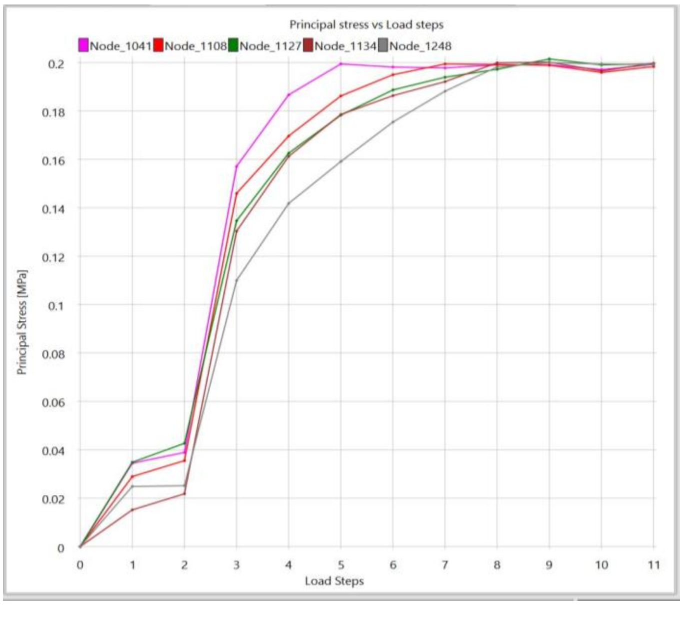

Figure 6.6: Principal stress vs Load step diagram for the monitoring points

Further deterministic models were prepared with the specified element nodes as 'monitor points' for principal stress. The material parameters were randomized in the software SARA-FReET to define the PDF of resistance of the structure to differential settlement.

6.3 Simplification of Numerical Model

Once the criterion for failure has been defined, the next step was to perform reliability analysis using a deterministic model with the parameters leading to the defined failure. Considering the high computational requirement for reliability analysis in SARA, certain assumptions and modifications were made to the numerical model. The geometry of the model was simplified as much as possible, without hampering the integrity of structural assessment. The foundations that has been added to the structure in the previous model was removed and just the superstructure was modelled. Besides, in order to reduce the number of finite elements in the mesh, the average mesh size was increased from 0.5 to 2.0 (Figure 6.7).

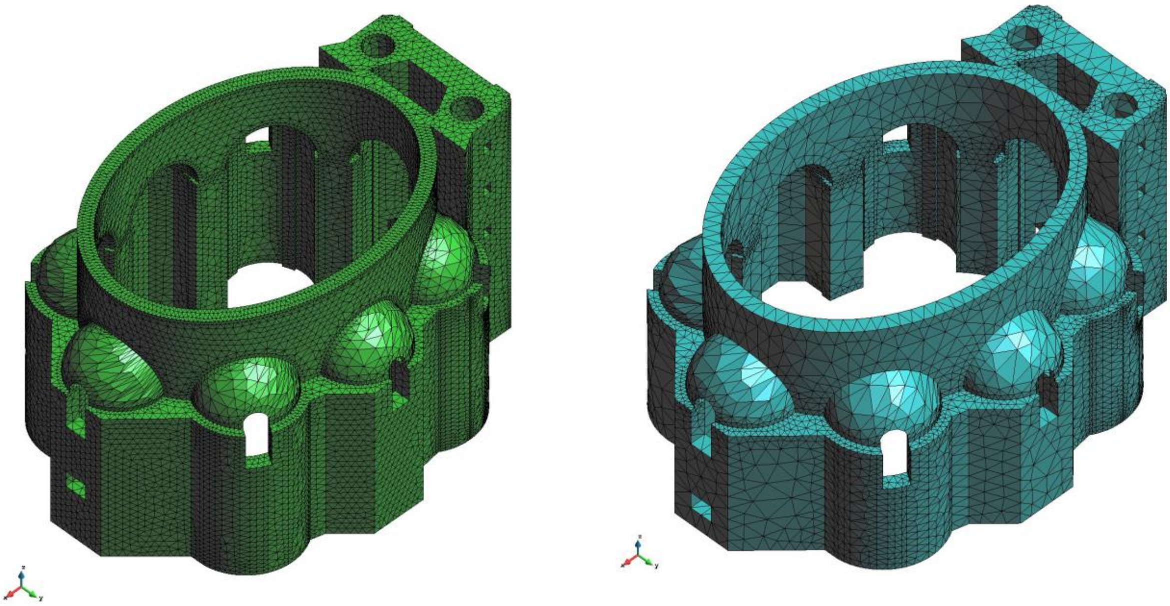

Figure 6.7: Numerical model before (a) and after (b) the modification of numerical model

For the modified numerical model, all load conditions, material parameters and boundary constraints has been kept the same. Moreover, the differential settlements has been added to the exact same location as the original model. However, the load step in the second interval (that comprises of differential settlement) was reduced from 10 steps to 8 steps so as to satisfy the failure criterion described in section 6.2. In this way, the modified numerical model was allowed a differential settlement of 80 mm (instead of 100 mm) distributed in 8 load steps. Hence, each load step corresponded to 10 mm of displacement.

Prior to perform a probabilistic analysis, it is very important to validate the results obtained in the deterministic analysis. To check this, the results obtained by the deterministic analysis of the modified numerical model was compared to the original one.

A significantly small difference in results, primarily the stresses and displacement were observed between the two models. However, the crack formation in the modified numerical were very similar to the original model.

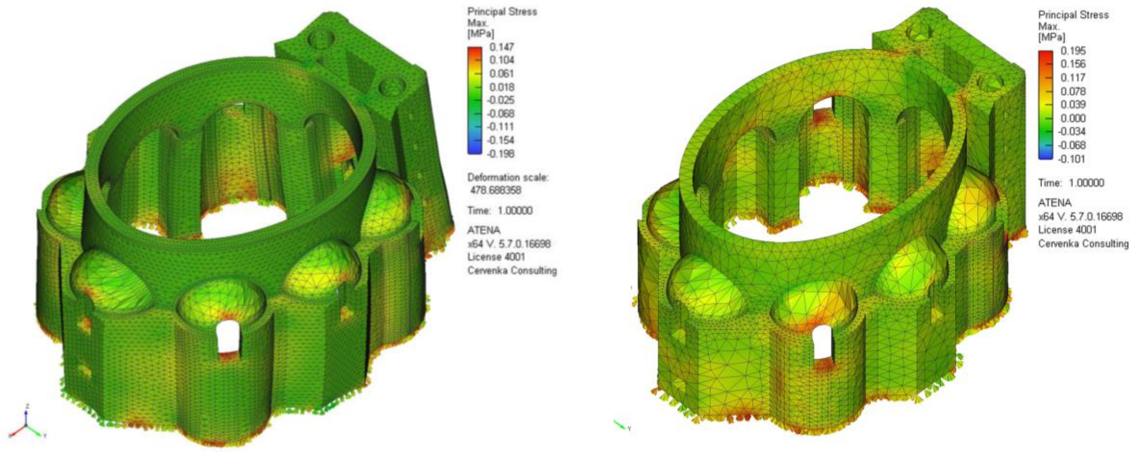

Figure 6.8: Principal stresses before (a) and after (b) the modification of numerical model

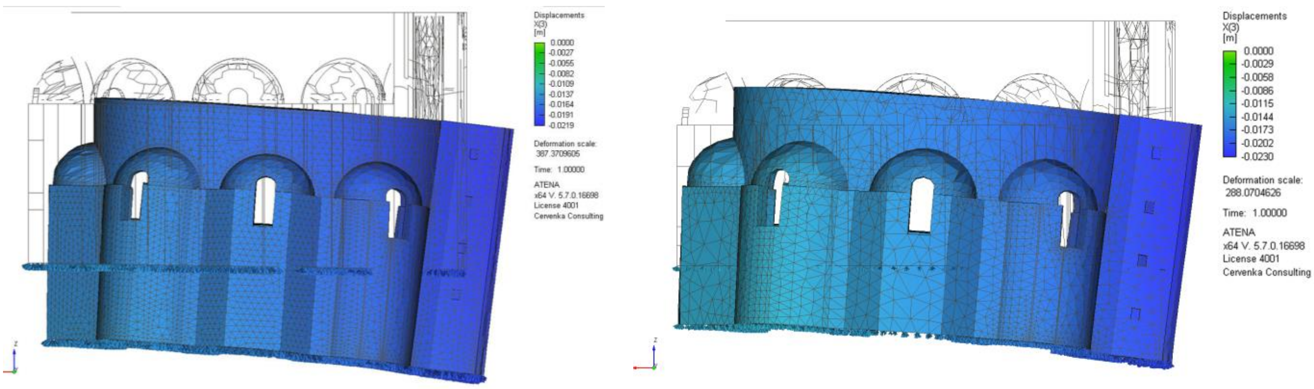

Figure 6.9: Maximum displacements in the structure before (a) and after (b) the modification of numerical model.

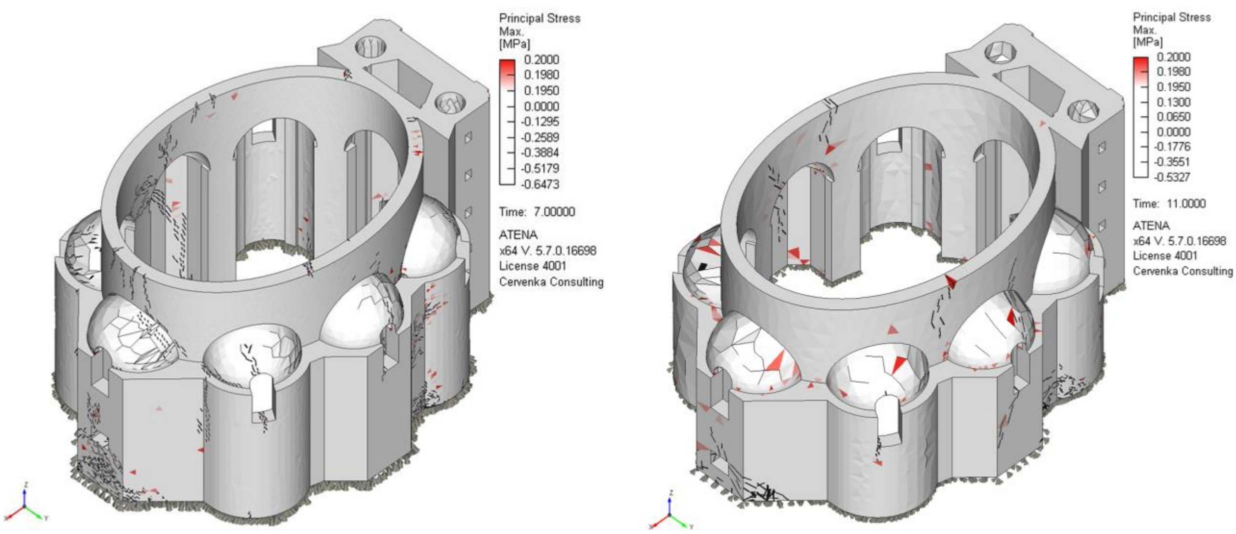

Figure 6.10: Development of cracks with the same load before (a) and after (b) the modification of numerical model.

Once the validation of the modified numerical model was accomplished, the monitor for principal stresses were assigned. The selection of points in the structure for monitoring the stresses was performed in a similar manner as explained in section 6.2. The tetrahedral elements in the mesh that reached the maximum principal stress in the last load step, i.e. when the failure criterion is achieved, has been chosen as monitor for the structure.

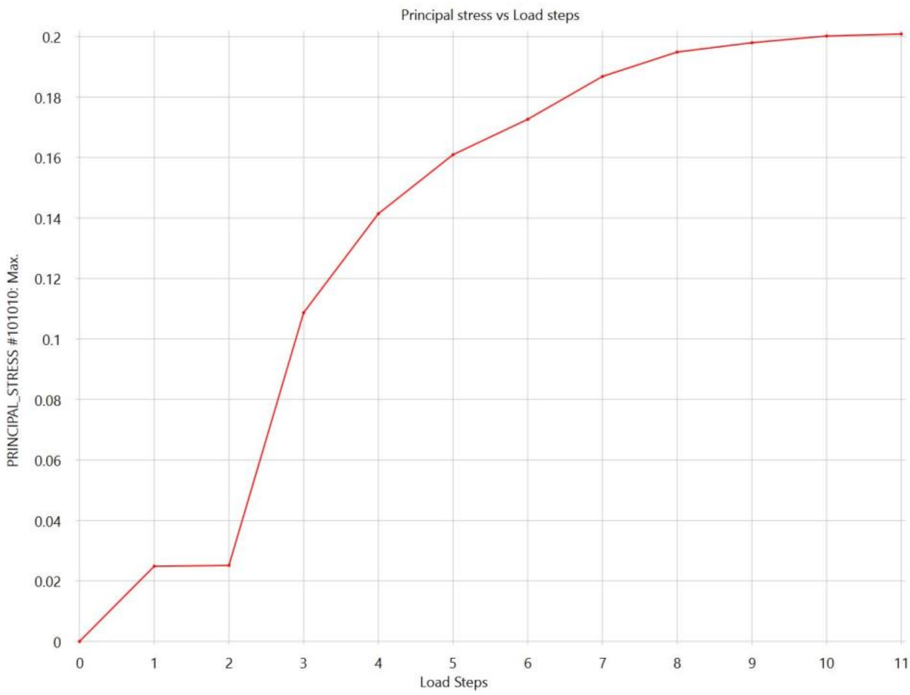

Figure 6.11: Stess vs load step graph for the selected monitor

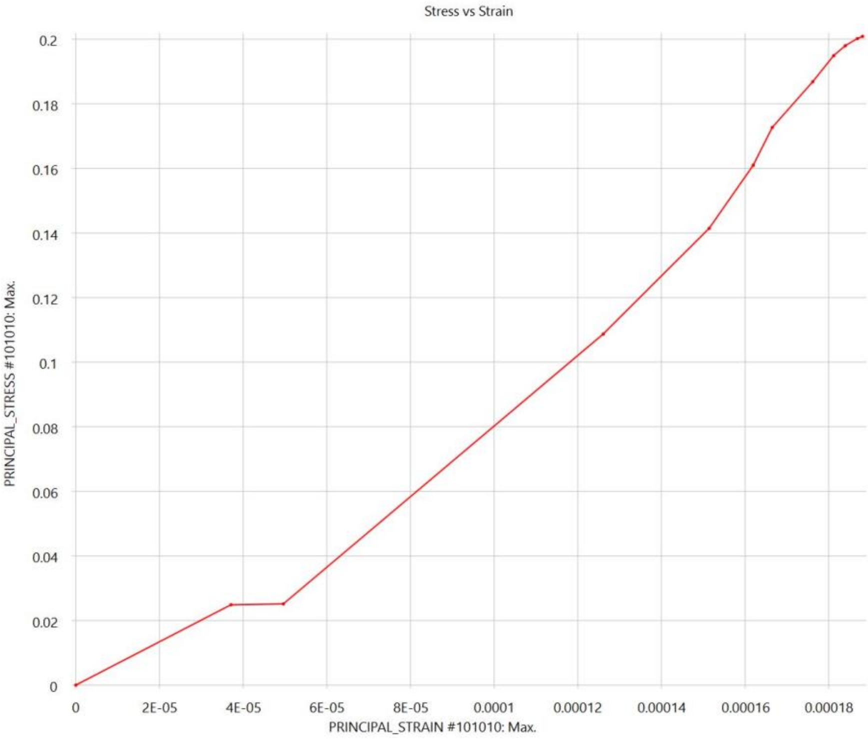

Figure 6.12: Stress vs strain graph for the selected monitor

6.4 Randomization of Input Parameters

The basic objective of statistical and reliability analysis is mainly to determine the statistical parameters of structural response and estimate the theoretical probability of failure. Stochastic or probabilistic finite element method used for the purpose accounts for the uncertainties in the geometry or material properties of the structure or the applied load. The uncertainties are modelled as random variables defined by their respective probability distribution function. An ideal case is attained when all random parameters are measured and there exists a real data. In that case, a statistical assessment of these experimental data such as the strength of material or loading, can be done and the most appropriate PDF (e.g. normal, Gaussian, lognormal etc.) can be selected (Freet program documentation). Prior to performing any probabilistic analysis, it is very important to identify the critical parameters and assume a random distribution for each of them. The different material parameters along with their corresponding mean value and coefficient of variation (COV) has been listed in table 2. The respective mean value is obtained from the deterministic analysis and the standard deviation has been taken from different literatures (Matos et al., 2010). Since in this case, there is no additional collected data, it is not possible to update the previous developed numerical model using inference procedure.

Table 2: Mean value, coefficient of variation and distribution function for the material parameters

Material Parameters		Distribution	Mean Value	COV
Modulus of Elasticity [GPa]	E_c	Normal	0.2	0.1
Compression strength [MPa]	f_c	Normal	2.9	0.2
Tensile Strength [MPa]	f_t	Normal	0.2	0.2
Tensile Fracture energy [N/m]	G_f	Normal	70	0.1

Once the variables has been defined, a correlation between those random input variables is introduced in the form of the correlation matrix. The correlation matrix for the variables has been obtained from literature and has been listed in table 3. Using values ranging from 0 to 1, a correlation among variables can be prescribed. It is important to note here that the correlation matrix must be symmetric and positive definite for the samples to be generated.

Table 3: Correlation matrix for the material parameters

	E_c	f_c	f_t	G_f
E_c	1.0	0.9	0.7	0.5
f_c	0.9	1.0	0.8	0.6
f_t	0.7	0.8	1.0	0.9
G_f	0.5	0.6	0.9	1.0

As a result of the above steps, a set of input parameters, comprising of the generated random variables described by their mean, variance and distribution function, is obtained for the ATENA computational model.

6.5 Simulation Technique

The simulation technique is one of the most widely used techniques to solve structural reliability problems. The fundamental idea underlying this technique is to simulate some phenomenon and then observe the number of times the specified events occurs. Although the concept is relatively straightforward, yet the process is computationally intensive.

6.5.1 Monte Carlo Simulation

The Monte Carlo method is special technique, that is used to generate some numerical results without performing any physical testing. In this technique. the output data obtained from the previous tests is applied to establish the probability distributions of the input parameters. Then, this distribution information is used to generate samples of the numerical data.

The Monte Carlo method is often used to solve complicated problems where closed-form solutions are either not possible or is difficult to obtain. Probabilistic problems that comprises of complex nonlinear finite element models can be solved using this technique, provided that the required input information and necessary computing power is available. Besides, it can also be used to solve complex problems in closed form by considering necessary assumptions. This technique facilitates to study the original problem without the selected assumptions to achieve more realistic results. Random input parameters are generated according to their PDF using Monte Carlo type simulation (Latin Hypercube Sampling).

6.5.2 Latin Hypercube Sampling

The major drawback of pure Monte Carlo simulation cannot be applied for time-consuming problems, as it requires large number of simulations (repetitive calculation of structural response) for analysis. This problem was partially solved using past studies, through approximate techniques suggested by many authors, e.g. Grigoriu (1982/1983), Hasofer & Lind (1974), Li & Lumb (1985), Madsen et al. (1986). Generally, the major issue of these techniques lies in the accuracy.

Therefore, studies were performed, focusing on the development of advanced simulation techniques, which concentrates simulation into failure region (Bourgund & Bucher 1986, Bucher 1988, Schuëller & Stix 1987, Schuëller et al.1989). Although some of the developed techniques usually require smaller number of simulations, as compared to pure Monte Carlo (thousands), an application for nonlinear fracture mechanics problem can be difficult and still almost impossible. However, there exists some feasible alternatives, such as Latin hypercube sampling (McKay et al. 1979, Ayyub & Lai 1989, Novák et al. 1998) and response surface methodologies (Bucher & Bourgund 1987), that can produce results with smaller number of simulations.

The simulation for the church was performed in the software FReET, which is based on the Latin Hypercube sampling process. Since the software has a very high computational need, the number of simulations were taken as 50, allowing not more than 0.001 deviation in the correlation matrix.

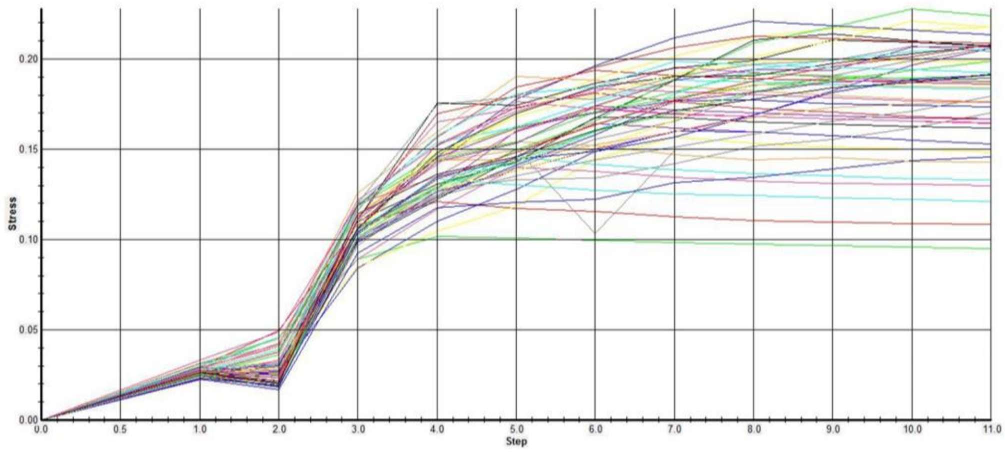

Figure 6.13: Stess vs load step graph of the structure obtained from randomizion of material parameters

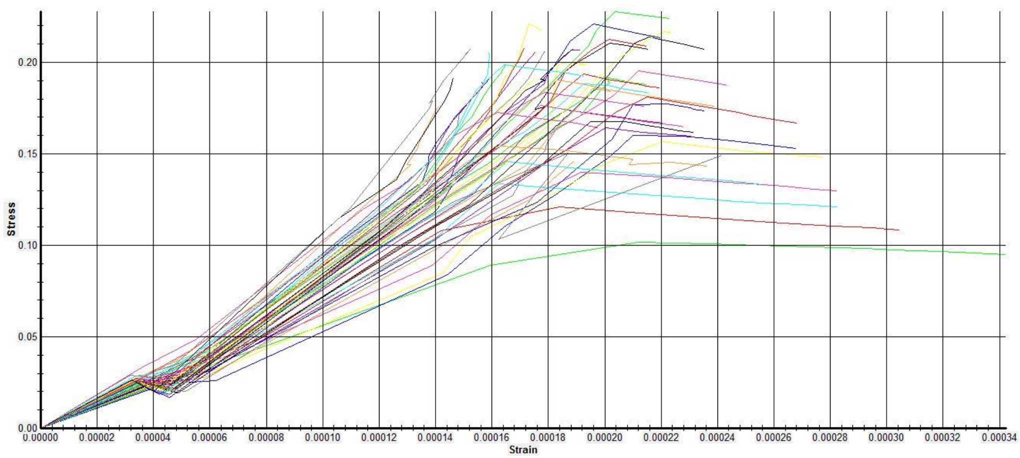

Figure 6.14: Stess vs strain graph of the structure obtained from randomizion of material parameters

The generated values of random parameters are then used as inputs for Atena computational model to perform deterministic analysis. The main objective of FReET- Atena basic statistical reliability nonlinear analysis is to get the estimation of structural response statistics (such as stresses, deflections, failure load etc), the sensitivity analysis and the assessment of reliability.

6.6 Sensitivity Analysis

6.6.1 General Methodology

The probabilistic based assessment of any structure may demand a lot of computational efforts, depending upon the chosen method, number of simulations and number of considered variables.

A sensitivity analysis allows to identify the parameters that are critical to the overall response function of the structure to the specified problem. The sensitivity analysis for St Barabara church was performed in SARA-FReET, which is a multipurpose probabilistic software used to determine the significance of the random variables. Novak et al 1993, provides a description of different methods used for sensitivity analysis. Based on Latin Hypercube Sampling (LHS), sensitivity analysis can be performed in two ways:

- In terms of the coefficient of variation
- In terms of the Non parametric Rank-Order Correlation

The first method listed above is based on the comparison of the partial variation coefficient of the response structure variables with the variation coefficient of the basic random variables.

SARA-FReET however, uses the second method listed above which incorporates the Non-parametric rank structural correlation obtained between the basis random variables and the structural response variable. The partial correlation coefficient obtained between the various input parameters and the response variable is used to measure the relative effects of each basic variable in the response function of the structure. The variables that has the most considerable influence are assumed to have a higher correlation coefficient (either positive or negative) than the other variables. In case of a weak influence, the correlation coefficient tends to be close to zero.

Latin Hypercube Sampling facilitates the application of sensitivity analysis to the structure, without any additional effort. This is because of the utilization of non-parametric rank order correlation to the nonlinear response of the structure. Basically, the nonparametric correlation considers the value of its rank among all other values, instead of the actual values in the sample set, to draw the resulting list of numbers from a perfectly known probability distribution function. Since the value of the random variable in case of LHS are not identical, there lies no necessity to consider the mid-rankings. When compared to the linear correlation, the nonparametric correlation is more precise, robust and distribution independent, which makes it appropriate for Latin Hypercube Sampling based sensitivity analysis.

For sensitivity analysis, SARA uses Spearman rank-order correlation coefficient and Kendall's tau. The Spearman rank order correlation coefficient is expressed as (Novak, 1998):

$$r_{s,i} = 1 - \frac{6\sum_{j=1}^{N}(q_{ji} - p_j)^2}{N^3 - N}, \qquad r_{s,i} \in [-1, 1]$$

where p_j is the rank of the ordered sample of response variable achieved as a result of j-th run of simulation process, $q_{i,j}$ is defined as the rank of any representative value of the random variable (X_i) in an ordered sample set of N simulated values, used in the j-th simulation.

Kendall's tau is defined by taking ½ N(N-1) pairs of N data points rank ($q_{j,i}$, p_j) as shown in the above equation. The data points when considered in either order is termed as one pair. A pair is known as 'concordant' (*c*) in case the relative ordering of the rank of two p_j is same as the relative ordering of the rank of two q_j. However, if the pairs are of opposite rank, it is called 'discordant' (*d*). Besides, the pair is called '*extra-p_j*' if the tie is in q_{ji} and vice versa. Using the above counts, Kendall's tau is expressed as:

$$\tau_i = \frac{c-d}{\sqrt{c+d+extra-p_j}\ \sqrt{c+d+extra-q_{ji}}}$$

Further, rank based statistics shows close relationship to parallel coordinates using which the correlation structure can be easily identified (Wegman,1990).

The sensitivity is represented graphically using cartesian and parallel coordinates representation. The rank p_j and q_{ji} are not represented orthogonally but are drawn in parallel and the pairs of points are joined by lines. Parallel coordinate representation provides a fine insight into the analysed problem. If the data is positively correlated, the lines do not tend to intersect between the parallel coordinate axes. In case of a strong negative correlation a near-pencil of lines is obtained. The result of sensitivity analysis is listed in figure 6.14.

#	Name	+ sensi ▽	— sensi	x -1
2	Masonry.Ft	0.67673		☐
1	Masonry.E	0.57916		☐
3	Masonry.Fc		-0.57637	☐
4	Masonry.Gf	0.54401		☐

Figure 6.15: Sensitivity Analysis- Influence of random variables to the calculated resistance

The influence of uncertainties is significant and the most important material parameters are recognized. Figure 6.14 shows results of sensitivity analysis based on non-parametric rank-order statistical correlation obtained between the individual random variables and the structural resistance. Random variables are ordered in the figure 6.14 according to the dominancy (positive or negative influence is described by positive or negative correlation coefficient). However, an exact measurement of these parameters should be performed in order to get realistic results.

6.7 Structural safety evaluation

6.7.1 Target Reliability

The reliability index β, can provide a quantitative assessment of the safety of the structure. Once the reliability of the structure has been determined, it should be subsequently compared to the base value or target reliability $\boldsymbol{\beta}_T$.

The different criteria for risk acceptance have been described in the Eurocodes in terms of target and acceptable (i.e. design) failure probabilities and their acceptable reliability indices. Further, they are used to calculate partial factors for design purposes. The values listed has been acquired from numerous studies conducted over the years by merging various approaches including optimization, calibration and human safety evaluation. The values indicate the possible consequences of failure, the reference time period and are applicable for component failures. However, special attention should be given while considering global failure conditions and target reliability criteria for existing structures.

The design failure probability is usually based on the expected economic and social consequences in order to portray the mentioned risk acceptance criteria. The EN 1990-2002 provides recommendations for the target reliability index for two reference periods (1 year and 50 years), without any explicit link to the design working life. The values are based on optimization and calibration and exhibits results from several studies.

Reliability classes	Failure conseq.	Reliability ind. 1 year	50 y.	Examples
RC3	high	5.2	4.3	bridges, public buildings
RC2	medium	4.7	3.8	residences, offices
RC1	low	4.2	3.3	agricultural buildings

Figure 6.16: Reliability classification for different reference period according to (EN 1990 2002)

It is important to note here that the couple of β value specified in figure 6.15 for each reliability class corresponds approximately to the same reliability level. The practical application of these values depends upon the time period that has been considered in the verification, as it may be associated with various statistical information concerning time variant actions (such as wind, earthquake, etc.) and the related vectors of basic parameters. For instance, a reliability index β=3.8 can be used for a structure of reliability class 2 (RC2) only when probabilistic models of basic variables are related to the reference period of 50 years. Besides, when β= 4.7 is applied using the theoretical model for one year, the same reliability level should be reached. It should be noted that the couples of β-value corresponds to the same reliability level provided the failure probabilities in the individual time intervals are independent.

Taking into account the reference period equal to the remaining working life, the reliability level corresponding to an arbitrary remaining working life can be derived using the following expression (EN 1990 2002):

$$\beta_{tref} = \Phi^{-1}\{[\Phi(\beta_1)]^{tr}\}$$

where β_1 is the target reliability index taken from figure 6.15 for a relevant reliability class and the reference period of one year. In case of model structure, it follows that target reliability index (β) of around 4.1 should be considered for a reference period of years

A more comprehensive recommendation has been provided by ISO 2394 1998, where the target reliability index is given for the working life and is related not only to the consequence of failure but also to the relative costs of the safety measures (figure 6.16). Accordingly, the target reliability $\beta \approx 3.1$ may be selected for model structures.

Relative costs of safety measures	Consequences of failure small	some	moderate	great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

Figure 6.17: Target reliability index (life-time, examples) in accordance with ISO 2394 1998

Similar recommendations for target reliability is provided by JCSS 2011, where the target reliability indices are also related to both the consequences and to the relative cost of safety measures. However, it takes into consideration the reference period of only 1 year.

Further, ISO 13822 2010 indicated four target reliability levels based on the different consequences of failure (the ultimate limit state) for the assessment of existing structures (Table 4). In this case, the related reference period is the minimum standard period for safety i.e. 50 years. Following the recommendations, the target reliability $\beta \approx 3.8$ can be assumed.

Table 4: Consequence of failure and target reliability (ISO 13822 2010)

Consequences of failure	**Target reliability index**
Small	2.3
Some	3.1
Moderate	3.8
High	4.3

6.7.2 Reliability Index

The reliability Index can be geometrically interpreted as the shortest distance from the origin of the reduced load and resistance distribution to the Limit State Function. It can be expressed as:

$$\beta = \frac{\mu_Z}{\sigma_Z}.$$

where μ_Z and σ_Z denotes the mean value and the standard deviation of the safety margin. The reliability index β can also be defined as the inverse of the coefficient of variation of the function g(R,Q) = R-Q where the variables R and Q are uncorrelated. In case of a normally distributed random variable R and Q, the probability of failure is related to the reliability index as (Novak, 2000)

$$\beta = -\Phi^{-1}(P_f) \quad \text{or} \quad P_f = \Phi(-\beta)$$

where Φ represents the standardized Gaussian distribution function.

Based on the above equation, a relationship between P_f and β can be established as expressed in figure 6.17.

P_f	β
10^{-1}	1.28
10^{-2}	2.33
10^{-3}	3.09
10^{-4}	3.71
10^{-5}	4.26
10^{-6}	4.75
10^{-7}	5.19
10^{-8}	5.62
10^{-9}	5.99

Figure 6.18: Reliability index and the probability of failure (Nowak, 2000)

To calculate the reliability index of St Barbara church, the stresses obtained in the monitor at different values of differential settlement were defined in the limit state function. The mean value in each case is equal to the maximum principal stress at that load step (figure 6.11). The value for the coefficient of variance for the load was taken as 0.2. The reliability index obtained for different level of differential settlement for the church of St Barbara has been listed in table 5.

Table 5: Reliability index for differential settlement of St Barbara church

Load Step	**Settlement**	**Type**	**Reliability Index**
1	18 mm	Uniform	5.8077
2	8 mm	Differential	5.4651
3	16 mm	Differential	2.1466
4	24 mm	Differential	0.9483
5	32 mm	Differential	0.4991
6	40 mm	Differential	0.2262

Comparing the obtained reliability index to the target reliability shows that a uniform settlement of 18 mm or a differential settlement of up to 8 mm is acceptable for the structure. Beyond the mentioned level of differential settlement, the structure approaches very closely to the failure condition.

6.7.3 Probability of failure

The probability of failure (P_f) of a structure can be obtained by considering the PDFs of the load (Q) and resistance (R) denotes as.

$$g\ (R,Q)= R-Q$$

When the load exceeds the resistance, the structure fails (Novak, 2000).

The probability of failure for St. Barbara church is calculated taking into account different scenarios as listed below. These results show that such probabilistic approach is desirable to assess seriously the reliability of historical structures. It should be noted that the term "failure" means here damage of a structural element (e.g. vertical crack in the enclosure wall, i.e. kind of serviceability limit state), and not the total collapse of the whole structure (ultimate limit state).

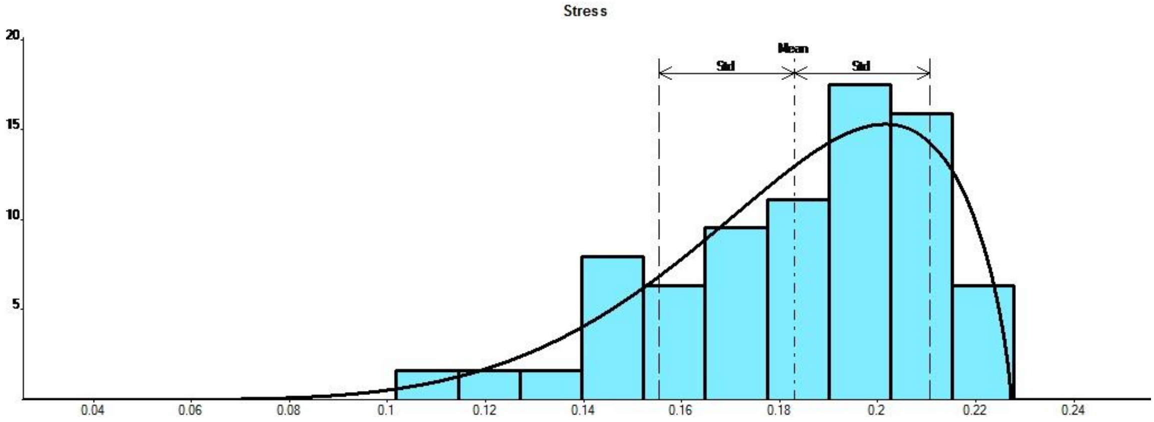

Figure 6.19: Histogram and the calculated probability distribution function of the structural resistance (uniform settlement)

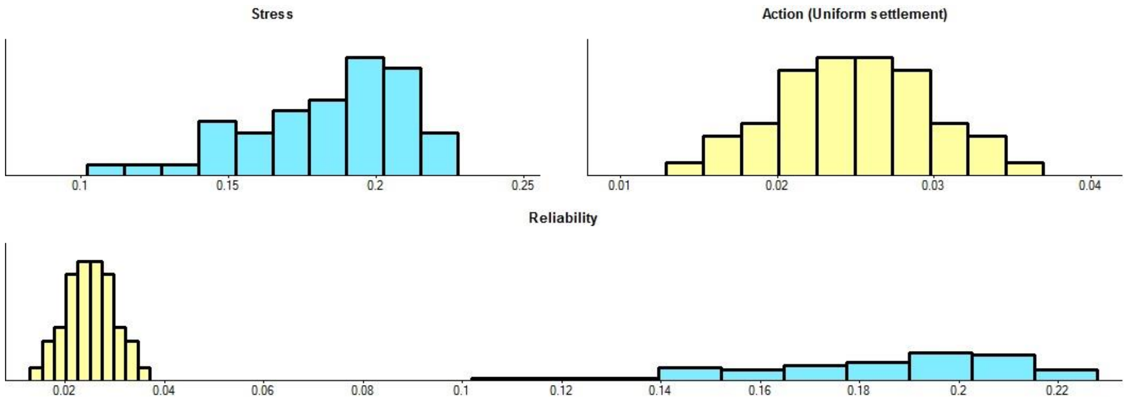

Figure 6.20: Graphical representation of the limit state function for probabilistic evaluation

The probability of failure of St Barbara’s church for the defined failure condition is listed in Table 6. Similar to the reliability index, the prabability of failure for each settlement has been computed from their respective stresses (shown in figure 6.11 a) using the SARA_FreET software. The table denotes an acceptable probability of failure for a uniform settlement or a differential settlement of up to 8 mm.

However, beyond 8 to 10 mm of differential settlement, the structure starts to portray high probability of failure.

Table 6: Probability of failure for differential settlement of St Barbara church

Load Step	Settlement	Type	Monitored Stress [MPa]	Reliability Index	Probability of failure
1	18 mm	Uniform	0.0249	5.5359	1.55E-08
2	8 mm	Differential	0.0251	5.3226	5.11E-08
3	16 mm	Differential	0.1087	2.1466	0.01591
4	24 mm	Differential	0.1414	0.9483	0.17149
5	32 mm	Differential	0.1609	0.4991	0.30886
6	40 mm	Differential	0.1726	0.2262	0.41054

Comparing the pattern and magnitude of cracks to the existing site conditions, it can be concluded that the structure is absolutely safe, representing probability of failure lower than 1.55E-08. The damages observed through visual inspection reveals that the settlement is almost neglegible.

7. CONCLUSION AND RECOMMENDATION

The reliability assessment of the church of St. Barbara, when compared to the existing condition of the structure, reveals that the structure is in a good condition with a low probability of failure due to differential settlement. However, it is important to emphasize upon the face that numerous assumptions were made while performing the analysis.

Material properties, which is the most important characteristic for any structure has been taken from the study conducted at St Ann church. Although both the churches are similar in terms of typology and geography, yet visual inspection shows that the St. Ann church is in a much-deteriorated condition as compared to St. Barbara's church. This makes the obtained results less accurate and conservative. Therefore, it is strongly recommended to perform tests to identify the actual material property of St. Barbara's church.

Besides, the structure needs to be constantly monitored to detect damages that could cause a risk to the structure. The condition for failure plays a very important role in reliability. Identification of the different failure mechanisms can be useful to determine the most probable cause of failure by using reliability techniques. Also, the major cracks that have been identified should be constantly monitored.

At present, the plinth stones show signs of damage. If the stone material is strongly damaged, the stability of the whole construction may be threatened. For example, in case of an increase of open porosity in the footing stones, especially of those pores where the capillary action takes place, the moisture can percolate into the upper part of the structure. This may not only hamper the aesthetic perception of the structure, but also the cohesion and durability of the stone masonry in the upper parts of the church. Consequently, it may reduce the strength on the walls. Hence, it is very important to pay attention to the survey of footing masonry and underlying rock conditions.

However unfortunately, this fact is not very often taken into consideration due to numerous reasons like lack of funds and difficulties in attaining appropriate samples, which would suit the specification and description for laboratory testing.

In addition, it is very important to note that the present codes and standards are mostly targeted to new buildings. This makes it difficult for its applicability in monuments and historical construction. Therefore, an upgradation in the existing codes can boost the application of reliability analysis to existing structures. Although reliability analysis is a very useful tool to precisely estimate the stability of structure, yet the huge computational cost, that it demands should not be neglected.

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8. REFERENCES

Ayyub, B.M. & Lai, K.L. 1989. Structural Reliability Assessment Using Latin Hypercube Sampling. In Proc. of ICOSSAR'89, the 5th International Conference on Structural Safety and Reliability, San Francisco, USA, August 7 - 11, Vol. I, Structural Safety and Reliability: 1177- 1184.

Bourgund, U. & Bucher, C.G. 1986. A Code for Importance Sampling Procedure Using Design Points - ISPUD - A User s Manual. Inst. Eng. Mech., Innsbruck University, Report No. 8 - 86.

Bucher, C.G. 1988. Adaptive Sampling - An Iterative Fast Monte - Carlo Procedure. J. Structural Safety, 5 (2): 119-126.

Cervenka, V., & Jendele, L. (2018). *ATENA Theory.* Prague.

Chodějovská, E., Semotanová, E., & Šimůnek, R. (2015). HISTORICAL LANDSCAPES IN BOHEMIA Regions of Třeboň, Broumov and Praha.

Facelli, G. (2014). Structural survey of St. Jacob's Church and coMParison with St. Ann's Church (Thesis).

Gajjar, P. (2018) Nonlinear numerical evaluation of the wall bearing capacity and the structure stability of the St. Ann Church from the Broumov Group of Churches, SAHC Master thessis, Czech Technical University in Prague.

Grigoriu, M. 1982/1983. Methods for Approximate Reliability Analysis. J. Structural Safety, 1: 155-165.

Hasofer, A.M. & Lind, N.C. 1974. Exact and Invariant Second-Moment Code Format. Journal of Eng. Mech. Division, ASCE, 100 (EM1): 111-121.

ISO2394:1998. General principles on Reliability for Structures. *International Standards Organization.*

Li, K.S. & Lumb, P.1985. Reliability Analysis by Numerical Integration and Curve Fitting. J. Struct. Safety, 3: 29-36.

Madsen, H.O., Krenk, S. & Lind, N.C. 1986. Methods of Structural Safety. Prentice – Hall, Englewood Cliffs.

Matos J. C., Cruz, P. J. S., Valente, I. B., (2010). UNCERTAINTY EVALUATION OF CIVIL ENGINEERING STRUCTURES BEHAVIOR.

Matos, J. C., Cruz, P. J. S., Valente, I. B., Neves, L. C., & Moreira, V. N. (n.d.). An innovative framework for probabilistic-based structural assessment with an application to existing reinforced concrete structures.

McKay, M.D., Conover, W.J. & Beckman, R.J. 1979. A Comparison of Three Methods for Selecting Values of Input Variables in the Analysis of Output from a Computer Code. Technometrics, Vol. 21: 239-245.

Milczarek, A. M. (2017). Anna Maria Milczarek Robustness-Based Assessment of Railway Masonry Arch Bridges.

Novák, D., Teplý, B. & Keršner, Z. 1998. The role of Latin Hypercube Sampling method in reliability engineering. In Shiraishi N., Shinozuka M., Wen Y.K., (eds). Proceedings of ICOSSAR-97 - 7th International Conference on Structural Safety and Reliability, Kyoto, Japan: 403-409. Rotterdam: Balkema.

Nowak A. S., Collins K. R., Reliability of Structures, United States of America: McGraw-Hill Higher Education, 2000.

Scacco, J. (2018). MASTER ' S THESIS PROPOSAL.

Schueller, G.I. & Stix, R. 1987. A Critical Appraisal of Methods to Determine Failure Probabilities. J. Struct. Safety, 4 (4): 293-309.

Schueller, G.I., Bucher, C.G., Bourgund, U. & Ouypornprasert, W.1989. On Efficient Computational Schemes to Calculate Structural Failure Probabilities. Probabilistic Engineering Mechanics, 4 (1): 10-18.

Schueremans L. (2001). Probabilistic evaluation of structural unreinforced masonry. Ph. D. Thesis„ KULeuven, Belgium.

Vicente N. Moreira et al, (2016). "Reliability-based assessment of existing masonry arch railway bridges," Construction and Building Materials, no. 115, pp. 544-554.

Wegman, E. J., "Hyperdimensional data analysis using parallel coordinates", Journal of the American Statistical Association, Vol. 85, No. 411, 664-675, 1990.

APPENDIX A

Table A: Borehole data in front of the church

Depth (in meters)	Description	Sample	Schmidt Reading
0,0 - 1,0	Red brown sandy soil with small stone particles		
1,0 - 1,2	Compact pieces of slightly weathered coarse-grained arcose sandstone		24
1.2	Thin layer of red brown soil with small fragments of fain grained clastic stones; easy to break down by hand		
1.4	Compact piece of brown reddish silty claystone	1	34.2
1,5 - 2,0	Small pieces of brown reddish silty claystone		
2,0 - 2,2	Big pieces of brown reddish silty claystone		
2.3	Pebble of hard claystone	3	
2.4	Sandstone masonry with mortar	4	
2,4 - 2,9	Fragments of coarse grained arcoes sandstone, easy to break down by hand; fragments of thick bedded gray claystone		
2,9 - 3,0	Sandstone masonry with mortar		
3,0 - 3,5	Small fragments of silty claystone, easy do break down by hammer	5	
3,5 - 3,7	Compact pieces of hard silty claystone		
3,7 - 3,9	Layer of soily character (weathered claystone) with small stone fragments		
3,9 - 4,5	Small fragments of silty claystone, easy to break down by hammer		
4.5	Compact piece of silty claystone		34.3
4,6 - 5,0	Small fragments of silty claystone		
5,0 - 5,4	Layer of strongly weathered claystone		
5,4 - 5,5	Layer of strongly disintegrated to soil character		
5,5 - 5,6	Compact piece of silty claystone		
5,6 - 5,8	Small fragments of silty claystone, easy to break down by hammer		
5,8 - 6,0	Compact piece of silty claystone		
6,0 - 7,0	Layers of weathered fine-grained sedimentary stone (claystone), easy to break down by hammer, easy to break		9.5

	down by hand, in some places harder positions		
7,0 - 7,3	Layer of thin bedded silty claystone, relatively hard, break down by hammer		
7,3 - 7,4	Layer of strongly weathered claystone		
7,4 - 8	Layers of weathered fine-grained sedimentary stone (claystone?), easy to break down by hammer, easy to break down by hand, in some places harder positions		
8,0 - 9,0	Compact pieces of silty claystone, in some places weathered layer		
9,0 - 10,0	Mostly weathered position, very easy to break down by hammer		
10,0 - 10,5	Small fragments of silty claystone		
10,5 - 12,0	Gray claystone, thin bedded, easy to break down by hand to thin layers	6	16

Figure 0.1: Layers of extracted soil from boreholes at the front of the church

APPENDIX B

Table B: Borehole data at the back of the church

Depth (in meters)	Description	Sample	Schmidt Reading
0,00 - 0,30	Brown sandy soil		
0,3 - 0,9	Fragments of coarse grained arcose sandstone (slightly weathered), the surface is easy to crumble by hand	10	18.7
0,9 - 1,0	Thin bedded silty claystone		
1,0 - 1,2	Bigger fragments of coarse-grained arcose sandstone (slightly weathered), the surface is easy to crumble by hand		
1,2 - 1,3	Layer of strongly weathered sandstone, disintegrated to sand		
1,3 - 1,5	Thin bedded silty claystone to siltstone	11	
1,5 - 2	Dark red brown sandstone, small fragments strongly weathered (extremely soft, disintegrated by hand), bigger fragments compact		23.5
1.8	Claystone		
2,0 - 2,5	Stone fragments with mortar		
2.5	Big pebble of strange stone material, not weathered, hard, probably andesite	12	
2,5 - 3,5	Weathered red brown silty claystone, small fragments	13	
3,5 - 4,4	Strongly weathered layer of soli character with claystone fragments		
4,4 - 4,6	Red brown claystone, easy to break down by hammer		21.5
4,6 - 5,0	Compact pieces of clay stones, red brown		
5,5 - 5,5	Slightly weathered layer of red brown claystone, easy to break down by hammer		
5,5 - 5,6	Layer of easy disintegrating clastic sediment of caly -silty character		
5,6 - 6,0	Bigger fragments of red brown claystone, some of them easy to break down by hammer		25.3
6,0 - 7,0	Sizely different fragments of easy break down red brown claystone		
7,0 - 7,4	Compact and harder fragments of claystone		
7,4 - 7,6	Layer of brown "soil" with small fragments of fine-grained sedimentary stones		

7,6 - 8,0	Compact fragments of claystone, red brown, easy to break down by hammer		28.4
8,0 - 8,15	Weathered grey brown clasic fain grained sediment (silty claystone), easy to break down by hand to soli character		
8,15 - 9,0	Layer of gray brown claystone, easy to break down by hammer	14	
9,0 - 9,5	Strongly weathered claystone, disintegrated to small fragments, easy to break down by hand		
9,5 - 10,0	Gray claystone, easy to break down by hammer		
10,0 - 10,5	Brown gray layer of clastic sediment, breaking down by hand to soil character		
10.5	Layer of harder claystone		
10,5 - 11,0	Light gray sediment of claystone character, easy to break down by hand, some harder possitions	15	
11,0 - 11,6	Layer of soft gray claystone, in some places easy to break down by hand to soli character		
11,6 - 12,0	Harder fragments of gray claystone		

Figure 0.2 Layers of extracted soil from boreholes at the back of the church